THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR Gordon Michael Babey

TITLE OF THESIS Calibration Of Robertson's Platoon Dispersion Model For Edmonton, Alberta

DEGREE FOR WHICH THESIS WAS PRESENTED Master Of Science In Civil Engineering

YEAR THIS DEGREE GRANTED Spring 1984

THE UNIVERSITY OF ALBERTA

Calibration Of Robertson's Platoon Dispersion Model For Edmonton, Alberta

by

©

Gordon Michael Babey

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF Master Of Science In Civil Engineering

Department of Civil Engineering

EDMONTON, ALBERTA

Spring 1984

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled Calibration Of Robertson's Platoon Dispersion Model For Edmonton, Alberta submitted by Gordon Michael Babey in partial fulfilment of the requirements for the degree of Master Of Science In Civil Engineering.

To

MARIE, MATHEW, AND JACQUELINE

ABSTRACT

The movement and dispersion of vehicles (cars, trucks and buses) as they travel between traffic signals is an important practical consideration in both the design and analysis of fixed-time signal co-ordination timing plans. Knowing the arrival pattern of a platoon of vehicles released from a previous set of traffic signals on a road is a requirement when calculating the signal offset and setting of a subsequent set of traffic signals on the same road. Platoon dispersion models provide some insight into this complex phenomenon.

Among the various studies in platoon dispersion (Lighthill's and Witham's wave theory, Pacey's diffusion theory and Robertson's recurrence model), the research by Robertson represents a major advancement. This is indicated by the incorporation of his platoon dispersion model in such signal optimization programs as TRANSYT (TRAffic Network StudY Tool) and the GLC (Greater London Council) Combination Method. Robertson's platoon dispersion model uses a recurrence exponential smoothing equation to predict a downstream arrival pattern (downstream cyclic flow profile) from an upstream arrival pattern (upstream cyclic flow profile). The smoothing factor is a function of the average travel time between the upstream and downstream cyclic flow profiles, and two empirical parameter: alpha and beta. Robertson suggests average numerical values for these parameters, but has qualified these values by stating that

the parameters may vary with the site characteristics.

This thesis concentrates on calibrating Robertson's Platoon Dispersion Model from the persective of vehicle delays, number of stops, and performance index for traffic platoons in Edmonton, Alberta. Actual real-time summer and winter platoon data was collected with a Micro-computer Cyclic Flow Profile Monitor.

The analysis indicated that the platoon dispersion predicted by Robertson's model using his recommended empirical parameters could be in error when compared with the observed data. The impact of this error on signal performance as quantified by delay, stops and performance index was significant. The analysis revealed that improper empirical parameters and the specific form of Robertson's model contributed to the error. Robertson's model requires the flow value in the previous downstream interval to predict the value in the present interval. This information is not available in computing the flow during the first iteration and is assumed to take a zero value. Since traffic platoons between fixed-time signals are cyclic in nature, and if steady state conditions exist, the flow for an interval will be the same over all cycle lengths. Based on this information an initialization equation was derived. For best results, the research indicates that calibration of the individual values of alpha and beta along with a modified form of Robertson's model is essential.

Based on the survey results there is some evidence to suggest that the individual values of alpha and beta are related to the degree of saturation on the street. The analysis suggests that as the degree of saturation decreases, the freedom to maneuver increases and hence the amount of platoon dispersion increases. The research provides average design guidelines for the alpha and beta empirical parameters during winter and summer weather conditions. Finally, the knowledge derived from using cyclic flow profiles is translated into several practical applications.

ACKNOWLEDGEMENTS

The Author would like to express his gratitude for the assistance provided throughout this research by numerous individiuals from the Civil Engineering Department of the University of Alberta, the Transportation Department of the City Of Edmonton, and the Transportation Engineering Group of Stanley Associates Engineering Ltd.

When the Author graduated from the University Of Alberta (1976), John Schnablegger P.Eng., encouraged him to seek higher levels of technical expertise by taking graduate level courses. He made it possible for the Author, while employed by the City of Edmonton to be the first person to use TRANSYT (TRAffic Network StudY Tool) Version 5 and later Version 6. TRANSYT Version 7 was used by the Author while employed by Stanley Associates Engineering Ltd.

The guidance, technical direction and inspiration provided by Professor Stan Teply P.Eng., throughout this research are gratefully acknowledged. Without this support the research objectives could not have been met. In addition, Professor Stan Teply made it possible for the Author to meet Dennis Robertson at the Transport Road Research Laboratory in England.

The author would like to thank Brad Shacker and Jim Chim for the time spent on the various computing program for data summary and analysis. Also to David Richardson, thank you for permitting the City of Edmonton and the University of Alberta to use an initial version of the ROBERT program.

Attahiru Alfa, gave necessary and patient assistance in explaining the proofs and basic theoretical assumptions which were required to expand on the strength of Robertson's platoon dispersion model.

Ed Foster, and his supervisor Orest Korbutt, made the Micro-computer Cyclic Flow Profile Monitor along with its supporting software packages available for this research.

The Author would also like to mention the numerous surveyors (Ed, Micky Ralph, Eunece, Hefny, Marie and Saleem) who participated in the data collection effort.

This research could not have been completed without an educational leave of absence, made possible by the Author's present supervisor, Geoff King. It is hoped that this research will contribute to the technical excellance that his team of signal designers continuously strive for.

Finally, I would like to thank my wife Marie, my son Mathew and my daughter Jacqueline for their patience during this research effort.

Table of Contents

Chapter Page

List Of Tables

List Of Figures

1. INTRODUCTION

1.1 Problem Statement

The movement and dispersion of vehicles (cars, buses and trucks) as they travel between traffic signals is an important practical consideration in both the design and analysis of fixed-time signal co-ordination timing plans. Knowing the arrival pattern of a platoon (an analysis of vehicular flow rates with a period equal to the cycle length, which indicates a higher rate of random flow, or which some type of regularity or pattern occurs, due to an upstream signal, may be defined as a platoon) of vehicles released from a previous set of traffic signals on a road is a requirement when calculating the signal offset and settings of a subsequent set of traffic signals on the same road. The application of this relationship is illustrated in Figure 1.1 and described below.

If the cumulative vehicular demand function (i.e., number of vehicles arriving at the signal approach) and the cumulative vehicular service function (i.e., number of vehicles discharging at the signal stop line) are plotted against time on the same graph, the following performance indicators can be quantified :

1. The area between the two cumulative functions represents the total vehicular delay,

2. A vertical line between the two cumulative functions represents the vehicular queue or the number of vehicles stopped at a given time, and

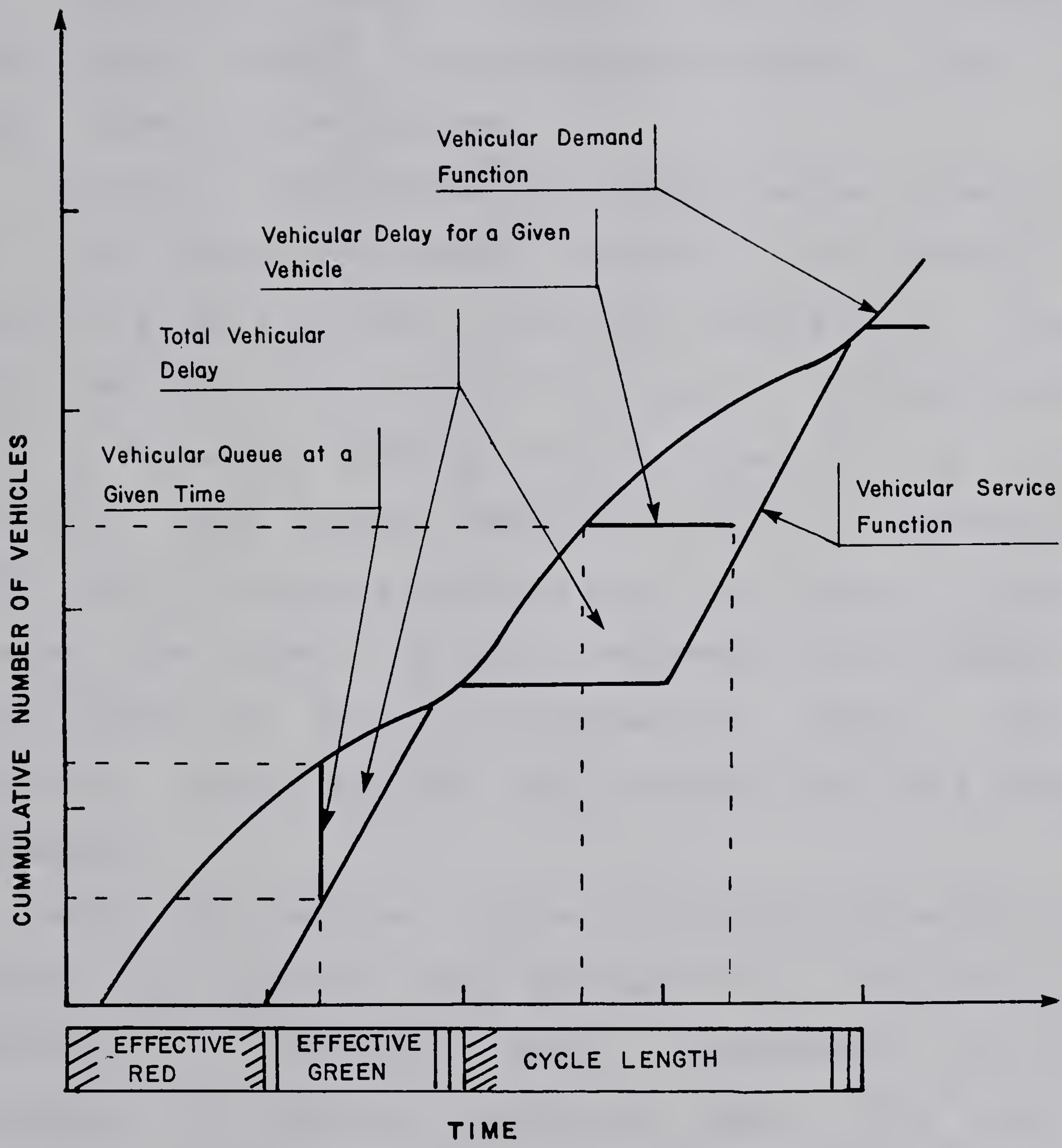


FIGURE 1.1

REPRESENTATION OF QUEUING AT A SIGNALIZED INTERSECTION

3. A horizontal line between the two cumulative functions represents the vehicular delay for a given vehicle.

The effect of varying the offset time between the two co-ordinated traffic signals at the end of the roadway link will be to move the cumulative service function relative to the cumulative demand function; thus the fundamental relationship between delay/stops/performance index and signal offset can be obtained.

Central to this fundamental concept is the observation that the cumulative demand function is influenced by the manner in which a platoon of vehicles moves along a roadway link. Vehicles in the platoon will attain different speeds due to a variety of factors: vehicle characteristics, driver behaviour, road surface characteristics, interference from pedestrians, roadside activities and side street traffic. Ignoring the effect of this phenomenon would reduce the effectiveness of signal coordination design. Platoon dispersion models provide some insight into this complex phenomenon.

Among the various studies in platoon dispersion, the research by Lighthill and Whitham[1,2], Pacey[3] and Robertson[12] represent major advancements in the development of platoon dispersion models. The work by Robertson is by far the most significant in terms of practical application, as indicated by the incorporation of his platoon dispersion model in such signal optimization programs as TRANSYT[13] and the GLC Combination Method[23]. The effectiveness of these programs for both conventional

and computerized signal control systems has been demonstrated in a number of field applications[24,25]. The significance of Robertson's research is partially due to his intoduction of the Cyclic Flow Profile Concept[26]. Robertson's platoon dispersion model is based upon the theory that the platoon of vehicles leaving an upstream intersection is formed by the queue of vehicles waiting at the start of green, departing at the saturation flow rate and dispersing as the platoon travels along the segment. This concept satisfies a practical need in that it represents a compromise between using micro-scopic models (simulation of individual vehicular behaviour) and macro-scopic models (relationships between vehicle flow rates and speed).

Continued research in the area of platoon dispersion has shown that the Robertson's model can provide a reasonable fit with observed data, but the accuracy of the model can be enhanced significantly by calibrating to local conditions and/or improving on the structure of the model. Table 1.1 provides a comprehensive and chronological summary of published calibrated empirical parameters in Robertson's platoon dispersion model. As shown in Table 1.1, there are a significant range of empirical parameters which result in a good prediction of the dispersion characteristics of observed traffic platoons. Lam [22] indicated that a lack of calibration may result in the prediction of minimum delay estimates which are in error in the range of 5 - 20%. It is

TABLE 1.1

COMPREHENSIVE CHRONOLOGICAL SUMMARY OF PUBLISHED CALIBRATED EMPIRICAL PARAMETERS IN ROBERTSON'S PLATOON DISPERSION MODEL

EMPIRICAL PARAMETERS Alpha (α)	Beta (β)	DESCRIPTION OF ROADWAY CHARACTERISTICS	SOURCE OF INFORMATION
0.50	0.80	Average of four sites based on 700 platoons. The sites were chosen with different physical characteristics ranging from single lane flow with heavy parking and restricted overtaking to a multi-lane site with no parking and free overtaking.	THE SYNCHRONISATION OF TRAFFIC SIGNALS FOR MINIMUM DELAY. Transportation Science. J.A. Hillier And R. Rothery, Volume I, 1967. (Original research for Robertson's Recommended Values).
0.40	0.80*	1. Three-lane dual-carriageway, with 10 to 15 percent of commercial vehicles in the peak hours and reasonable freedom for overtaking.	ANOTHER LOOK AT PLATOON DISPERSION (NO. 3 The Recurrence Relationship). Traffic Engineering And Control. P.A. Seddon, February 1972.
0.63	0.80*	2. A two-way road of overall width of 10.7 metres with 2 to 3 percent trucks and buses in the peak hour and limited freedom for overtaking.	
		Note: Both studies represent average values for 5 points where the distance between upstream and downstream measurements was at 61, 122, 183, 244 and 305 metres.	
0.24	0.80	Average values for a typical suburban roadway with two lanes in each direction. With the provision of turn pockets, it is relatively free of friction caused by turning traffic. Details of the individual measurements follow for one site:	STUDIES OF A PLATOON DISPERSION MODEL AND IT'S PRACTICAL IMPLICATIONS. Proceedings Of The Seventh International Symposium On Transportation And Traffic Theory (Kyoto). J.K. Lam, August 1977.
(.34/.15/.15)	(.75/.77/.79)	152 metres apart, (AM/OFF/PM) Peak Period	
(.23/.11/.38)	(.82/.76/.67)	341 metres apart, (AM/OFF/PM) Peak Period	
(.21/.12/.38)	(.79/.76/.70)	457 metres apart, (AM/OFF/PM) Peak Period	
(.42/.35/1.00)	(.71/.61/.52)	188 metres apart, (AM/OFF/PM) Peak Period	
(.15/.26/0.11)	(.80/.66/.88)	305 metres apart, (AM/OFF/PM) Peak Period	
(.18/.17/0.30)	(.75/.75/.78)	116 metres apart, (AM/OFF/PM) Peak Period	
0.33	0.80*	Heavy commuter traffic and few turning movements. Roadway classified as C.B.D. consisting of many small commercial businesses. Distance between upstream and downstream measurements was 385 metres.	A STUDY OF THE EFFECTIVENESS OF THE TRANSYT SIMULATION MODEL. Prepared For: Civil Engineering Department, University of Waterloo. Prepared By: P.E. Grubb, December 1977.

Continue with TABLE 1.1

EMPIRICAL PARAMETERS Alpha (α)	Beta (β)	DESCRIPTION OF ROADWAY CHARACTERISTICS	SOURCE OF INFORMATION
0.60 0.70	0.63 0.59	Single carriage way, 10.0 metres in width, with a gradient of approximately 5 percent. The road is subject to a 30 mph speed limit and clearway regulations apply during peak periods. Bus volume was 17 buses per hour. Distance between upstream and downstream measurements for each survey was 420 metres and 560 metres respectively.	PLATOON DISPERSION ALONG A MAJOR ROAD IN SHEFFIELD. Traffic Engineering And Control. T.Y. El - Reedy And R. Ashworth, April 1978.
0.63	0.80 *	1. Combination of parking, moderate to heavy turns, moderate to heavy pedestrian traffic and narrow lane widths. Traffic flow typical of urban Central Business District (C.B.D.).	GUIDELINES FOR SELECTING TRAFFIC SIGNAL CONTROL AT INDIVIDUAL INTERSECTIONS (Volume I). Prepared For: National Cooperative Highway Research Council. Prepared By: Alan M. Voorhees and Associates, Georgia Institute of Technology, July 1979.
0.46	0.80 *	2. Light Turning traffic, light pedestrian traffic, 3.35 - 3.66 metre travel lanes and possibly divided. Typical of well designed C.B.D. arterial.	
0.30	0.80 *	3. No parking, divided turning provisions and a 3.66 metre lane width. Suburban high-type arterial.	
		Some selected arterial roads in Kuwait.	PRELIMINARY SPECIFICATION AND SYSTEM DEFINITION. Area Traffic Control Study, Part 1, Phase 1. Prepared For: Government of Kuwait, Kuwait Municipality, Ministry Of Interior. Prepared By: Wilbur Smith And Associates, February 1981.
0.43	0.80 *	168 metres apart	
0.37	0.80 *	430 metres apart	
0.43	0.80 *	551 metres apart	
0.37	0.80 *	953 metres apart	
0.37	0.80 *	953 metres apart	
0.62	0.80 *	1,100 metres apart	
0.50	0.80 *	1,343 metres apart	
0.62	0.80 *	2,200 metres apart	

* INDICATES THAT THE VALUE WAS ASSUMED AND NOT CALIBRATED

this variablility in the empirical parameters and the impact of these empirical paramaters on such signal performance indicators as delay, stops and performance index that has prompted this research for the Edmonton, Alberta environment. More specifically, this research effort concentrates on the dispersion of traffic along roadway links typically in the Central Business District.

In addition, cyclic flow profiles provide the transportation engineer with a great deal of additonal information relevant to signal design and analaysis. Such profiles allow: review of average flow conditions, identification of major/minor road inflows, the benefits of signal co-ordination, estimates of saturation flow, estimates of spare capacity, estimates of start and end lags, estimates of platoon speed and amount of dispersion, incident detection possiblities, verification of signal settings by TRANSYT, possibilities for an international exchange standard, determination of preferential street treatment and ranking of the importance of network links. They also point to the need for coordination, and aid in computer network condensation. The use and understanding of this additional information is essential and a pre-requisite to good signal design and analysis.

1.2 Thesis Objective

The primary objective of this thesis is to calibrate Robertson's Platoon Dispersion Model for data in Edmonton, Alberta. Data was based on real-time traffic platoons during summer and winter conditions. This data base was in response to the practical problems associated with winter driving conditions in Alberta [33] and more generally across Canada. To achieve the objective two tasks were undertaken. The first task involved the validation of the basic structure of the model as proposed by Robertson and/or some modification to its basic form based on a mathematical extension. The second task involved the calibration of the model(s) to several "typical" roadway links which characterize arterials in the Central Business District. The research objective requires the following outputs:

1. Documentation of the difference in signal performance characteristics such as delays, stops and performance index between a calibrated and not calibrated platoon dispersion model.

2. Presentation of design guidelines which will assist in the selection of the most appropriate empirical parameters.

A secondary objective of this reseach is to translate the experience obtained during the course of this reseach into a series of practical applications as outlined in Section 1.1.

1.3 Scope Of The Research

In organizing this research project a number of activities are defined which will dictate the course of action in meeting the research objectives. The following considerations will assist in limiting the research to a manageable level:

1. This study will not attempt to verify and calibrate any platoon dispersion model other than that suggested by Robertson.

2. All street segments to be surveyed will be operated under a fixed time signal control strategy.

3. The selected survey locations (total of 6 measurements during the A.M. peak hour) will represent typical arterial roadways within or leading to the Central Business District (CBD) in Edmonton, Alberta, Canada.

4. Dispersion analysis will be restricted to link lengths which are typical of the CBD environment.

1.4 Organization Of The Thesis Document

The introduction along with the problem statement, thesis objective, scope of the research and organization of the thesis document are given in Chapter 1.

Chapter 2 reviews, in detail, past research efforts in the area of platoon dispersion. Thoeretical aspects of Lighthill's and Whitham's Wave Theory, Pacey's Platoon Diffusion Theory and Robertson's Recurrence Model are presented. Mathematical considerations in Robertson's Recurrence Model are critically discussed.

Chapter 3 discusses the data collection efforts with respect to winter and summer platoon data and speed/travel time data. In addition, the survey sites and their observed A.M. peak hour characteristics and site conditions are presented.

Chapter 4 presents the basic analytical tools that were used in the analysis process. More specifically, three computing programs were employed to aid in the determination of the "best" empirical parameters.

Chapter 5 presents a detailed summation of all analysis undertaken. The results of the calibrated process are discussed from the perspective of delay, stops and performance index. Design guidelines are also presented to assist in the selection of the most appropriate empirical parameters in Robertson's Model. In addition, practical applications of observed traffic platoons are presented.

Chapter 6 presents the conclusions, recommendations and further research suggestions arising from this research effort.

2. PLATOON DISPERSION MODELS

2.1 Overview Of Past Research Efforts

The amount of theoretical and applied research into the behaviour of traffic platoons and effects of their dispersion on traffic control has increased steadily since 1955.

One of the earliest (1955) research works related to platoon dispersion was carried out by Lighthill and Whitham([1,2]). These scientists presented their kinematic wave theory in two papers, the former giving a more mathematical treatment and dealing specifically with flood movement in long rivers, the latter giving a more descriptive treatment and dealing with the flow of traffic on long crowded roads. In the two papers, attention was drawn to waves which can be expressed in a one-dimensional flow system. These waves exist if, to a sufficient approximation, there is a functional relationship between the flow or volume, the concentration or density and the distance along the roadway link. On the assumption of a functional relationship, the wave theory follows from the equation of continuity alone. Accordingly, the waves are described as "kinematic", as opposed to "dynamic" waves which depend on Newton's second law of motion and some other assumptions relating stress to displacement, strain, or curvature. Kinematic waves do not disperse as other waves do, but they suffer a change in form, due to the dependence

of the wave speed on the flow carried by the waves. Accordingly, kinematic wave forms may develop discontinuties due to the overtaking of slower waves by faster ones. These were described as "shock waves" since they were formed in the same way as shock waves in a gas. From a plot of both the continuous kinematic waves and shock waves, the cyclic flow profile, and thus the platoon structure at any point along the roadway section, could be derived.

Pacey(3), in 1956, attempted to use Lighthill and Whitham's kinematic wave theory to predict platoon dispersion, and also developed his own platoon prediction model by treating platoon dispersion as a fluid diffusion process. The fluid diffusion theory assumes that the only changes in shape of the bunch of traffic released from the signals arise from differences in speed between vehicles in the bunch; there is no interference with overtaking, and any given vehicle in a platoon proceeds with a constant velocity (normally distributed about a mean speed) irrespective of the number or distribution of the vehicles on the road.

Lewis(4), in 1958, carried out a number of surveys downstream from a signal on a semi-expressway facility and found that signal coordination is feasible for a link distance of up to 1.0 kilometre (1,000 metres) or 0.65 miles (3,430 feet).

Gerlough(5), in 1961, suggested that an analogy exists between traffic dynamics and wave mechanics which could be used to describe the dissipation of traffic platoons when

released from a traffic signal.

Graham and Chenu[6], in 1962, studied the phenomenon of platoon dispersion at various points downstream from a signal. Field studies confirmed that at points 405, 805, 1210, and 1610 metres (1/4, 1/2, 3/4 and 1.0 mile), the percent of vehicles remaining in the platoon was 91, 85, 80 and 77%, respectively. This applied research confirmed the basic weakness of co-ordination design methods based on maximizing bandwidth.

Grace and Potts[7,8] and Herman et al[9,10],in and around 1964, gave a more mathematical treatment to Pacey's diffusion theory and showed that the model can be described by a one-dimensional pseudo-diffusion equation.

Hillier and Rothery[11], in 1967, concluded that platoon dispersion can be taken into account in the design of co-ordination and that the offset setting for minimum delay at the downstream signal up to 305 metres (1,000 feet) is a linear function of the distance from the upstream signal.

Robertson[12], in 1967, introduced a recurrence relationship to predict platoon dispersion in his contribution to the discussion on Area Control Of Road Traffic. This work was carried out at the Road Research Laboratory and is further reported[13,14,15] in the description of the TRANSYT (TRAffic Network StudY Tool)method of co-ordinating traffic signals. The model uses a recursive smoothing equation and is iterative in nature.

Although it appears that Robertson devised the method empirically it has been shown elsewhere[16] that Robertson's formula is similar to that of Pacey's, except that Pacey's transformed, normal distribution of journey time is replaced by a geometric distribution.

Seddon[17,18,19] in 1971/72 attempted to validate the three basic platoon prediction models proposed by Lighthill and Whitham, Pacey, and Robertson, respectively. He found that the Lighthill's and Whitham's theory could provide reasonable platoon prediction if an accurate flow-concentration relationship is available. Pacey's model preformed well for medium traffic but was not as accurate as Lighthill's and Whitham's in the case of heavy traffic conditions. For best results, it was concluded that all models should be calibrated for field conditions.

Hartley and Powner[20], and Tracz[21], in 1971, suggested that a rectangular distribution of journey time gives a similar predicted arrival pattern to the transformed normal or geometric distribution (i.e. Pacey or Robertson). This is important because it would make the prediction of platoon dispersion even faster than by use of the Pacey or Robertson model.

Lam[22], in 1977, presented the results of experimental studies on the accuracy and sensitivity of Robertson's model, and also its calibration against actual field data in Toronto. The results indicated that platoon data predicted by Robertson's model provided a reasonable fit with the

observed data, but the accuracy could be improved by the introduction of a special equation to initialize the platoon prediction process. Without the initialization equation, there could be significant errors in the predicated platoon leading edges, thus leading to sub-optimal co-ordination designs.

Among the various studies in platoon dispersion, the research by Lighthill and Whitham([1,2]), Pacey([3]) and Robertson([12]) represent major advances in the development of platoon dispersion models. The work by Robertson is by far the most significant in terms of practical application. This is indicated by the incorporation of his platoon dispersion model in such signal optimization programs as TRANSYT ([13]) and the GLC Combination Method([23]). The effectiveness of these programs for both conventional and computerized signal control systems has been demonstrated in a number of field applications([24,25]), and this effectiveness is partially due to the accuracy of Robertson's platoon model in describing traffic flow patterns in a signal network.

2.2 Theoretical Aspects Of Three Basic Platoon Dispersion Models

2.2.1 Lighthill's and Whitham's Wave Theory

Lighthill and Whitham([1,2]) put forth the theory that kinematic waves exist in a traffic platoon (or hump as they described it) if there exists a functional relationship

between the following explanatory variables:

$$q = uk$$

where : q = flow (vehicles passing a given point in units of time)

k = concentration (vehicles present per unit length of road), and

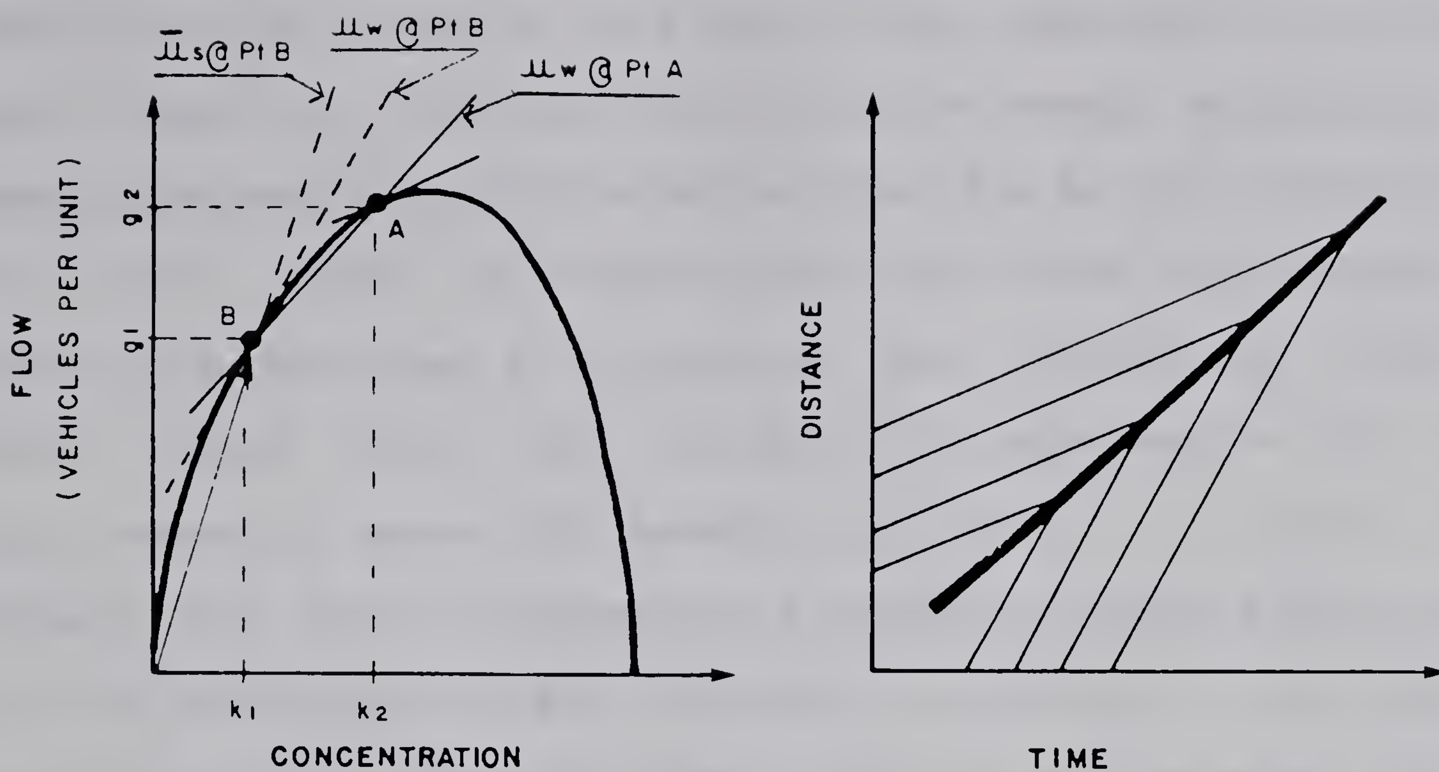
u = speed (along the road).

The approach unifies speed-flow relationships at low concentrations and headway phenomena at high concentrations. On the assumption of a functional relationship, the wave theory follows from the equation of continuity alone. Accordingly, the waves are described as "kinematic" (pure motion without reference to the masses or forces involved) as opposed to "dynamic" (motion and the equilibrium of systems under the action of forces) waves which depend on Newton's second law of motion (mass times acceration equals force).

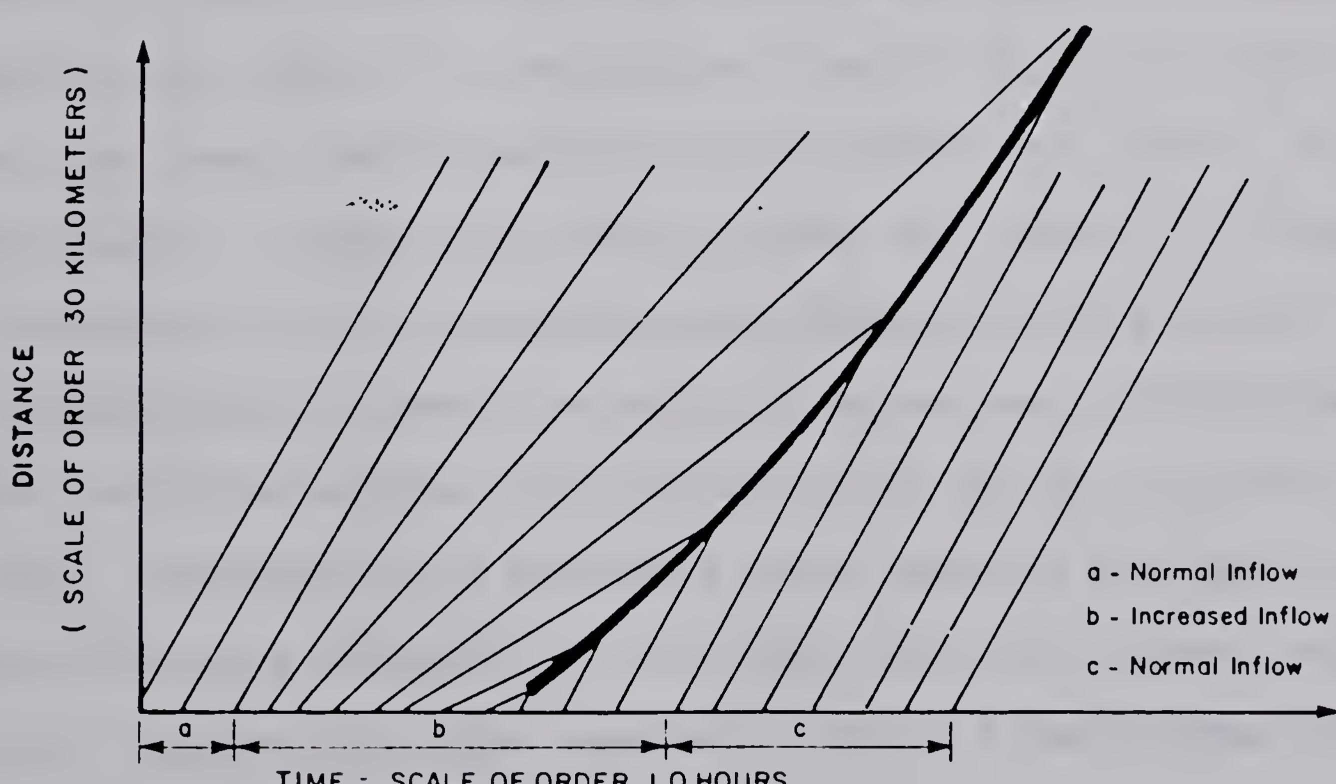
Several characteristics of the flow-concentration (q - k) curve, as defined by Lighthill and Whitham, are illustrated graphically in Figure 2.1 and are summarized below:

1. For any point on the flow-concentration curve, the radius vector represents the traffic speed ($\overline{u}_s$).

2. The tangent to this point on the flow-concentration curve represents the wave velocity (u_w). It is given by differential of q with respect to k (dq/dk) at that point. This slope is smaller than the traffic speed ($\overline{u}_s$), provided that the traffic speed decreases with an increase in concentration. This is acceptable for crowded roads and means that disturbances are propagated backwards to the stream of vehicles.



ANALYSIS OF TRAFFIC SHOCK WAVE USING A FLOW-CONCENTRATION CURVE AND A TIME-DISTANCE DIAGRAM. THE WAVE SPEEDS IN THE TIME-DISTANCE DIAGRAM ARE DRAWN PARALLEL TO THE RESPECTIVE TANGENTS (uw @ Pt. B AND uw @ Pt. A) OF THE FLOW-CONCENTRATION DIAGRAM.



PROGRESS OF A TRAFFIC HUMP WITH TIME

FIGURE 2.1

LIGHTHILL'S AND WHITHAM'S WAVE THEORY

Kinematic waves do not disperse as other waves do, but they suffer a change in form due to the dependence of the wave speed on the flow carried by the waves. Accordingly, wave forms may develop discontinuities due to the overtaking of slower waves by faster ones (with lower flow values). These were described by Lighthill and Whitham as "shock waves" since they were formed in the same way as that of shock waves in gases. For example, on the q - k curve in Figure 2.1, point A represents a condition where traffic is flowing near capacity and the speed is reduced to a value well below the free-flow speed. Point B, on the other hand, represents a condition where traffic flows at a somewhat higher speed because of the lower density. Tangents at points A and B represents the wave velocities for these two conditions. Now, if the faster flow of point B occurs later in time than that of point A, the waves of point B will eventually catch up with those of point A. This is illustrated in the time-distance diagram of Figure 2.1. The intersection of these two sets of waves has a slope equal to the chord connecting the two points on the q - k curve, and this intersection represents the path of the shock wave. Note that the velocity of the shock wave is often negative with respect to the roadway and is always negative with respect to the traffic.

The law of motion of kinematic shock waves is derived from considerations of conservation of vehicles, as was the law of continuous waves. If the flow and concentration

values (q_1, k_1) on one side of the shock waves, and the flow and concentration (q_2, k_2) on the other side of the shock wave, both which move with speed (v), then the quantity crossing per unit time may be written as :

$$q_1 - vk_1 = q_2 - vk_2$$

This gives the velocity of the shock wave as :

$$v = (q_2 - q_1) / (k_2 - k_1)$$

The velocity of the shock wave, by definition, is the slope of the chord joining the two points on the q - k curve (for a given value of x), which corresponds to the states ahead of, and behind the shock wave when it reaches x.

The waves on the time-distance diagram as presented are not trajectories of vehicles, but lines of constant flow and thus lines of constant speed. The individual vehicles have a greater velocity than the waves, because the speed of vehicles stream is represented by the radius vector, whereas the velocity of the waves is represented by the tangent.

Lighthill and Whitham applied such analysis techniques to the progress of a traffic "hump", a hump being a parcel of increased density, such as might occur on a freeway, flowing at a constant level along the main stream, where there is a short-term influx of substantial proportions at an on-ramp. Applying such analysis, traffic waves associated with the formation of a hump could be determined. The speed of the front of the hump can be obtained immediately from Figure 2.1, as the velocity of the earliest wave

representing the increased flow. The construction of the path of the shock wave, as discussed earlier, is also shown in Figure 2.1.

In summary, to obtain the kinematic wave and shock wave speeds, a detailed knowledge of the q - k relationship for the roadway in question is necessary, and this relationship has to be assumed constant over a given length of road at a given time. This assumption would seem reasonable provided no change in the composition of the traffic stream (i.e. the proportion of slower moving trucks and buses) takes place during the period considered. Clearly this composition and the motivation of drivers is likely to change during the day, and hence different q - k relations would be needed. However, during a given peak hour, it seems reasonable to assume that it would remain constant. From a plot of wave paths of both the continuous waves and shock waves, flow/time relationships, and thus the platoon structure at any point along a road can be determined. It thus appears that, provided the various assumptions are acceptable, the Lighthill's and Whitham's wave theory offers a method of predicting the behaviour of platoons as they progress along the road from traffic signals.

The following are the three major criticisms of the Lighthill's and Whitham's kinematic wave theory:

1. The theory requires an accurate model of q - k relationships which is difficult to obtain, especally for urban arterials where stop-start conditions prevail.

2. The predicted platoon does not increase in length as such a platoon does in reality, due to a weakness in the theory.

3. The plotting of the various wave paths is a very time consuming process.

As far as the Researcher knows, the Lighthill's and Whitham's wave theory has not been used in any signal control design procedure to date.

2.2.2 Pacey's Platoon Diffusion Theory

Pacey(3) assumed that there is no interference with overtaking (passing is possible at will), and that any given vehicle proceeds with the same velocity irrespective of the number or distribution of vehicles on the road. This is likely to be the case where flows are light on wide roads, but it is unlikely that free overtaking will be possible in congested urban conditions.

Assuming the distribution of velocities of vehicles to be normal, it is possible to evaluate the distribution of journey times between the two observation points. Pacey's platoon diffusion may be descibed as follows: Let

$f(u)du$ = the probability distribution of platoon speeds, assumed to be a normal distribution,

τ = travel time between the two observation points,

= D/u where D = distance between the two observation points, and

$g(\tau)d\tau$ = distribution of travel times.

The probability that a vehicle travel time lies between τ and $(\tau + d\tau)$ is the same as the probability that the corresponding speed lies between u and (u - du). Noting that
$u = D / \tau$,
$du = (D / \tau^2) d\tau$, and
$g(\tau)d\tau = f(u)du = f[(D/\tau) (D/\tau^2) d\tau]$.

Now, because speeds are assumed to be normally distributed,

$$f(u) = 1/(\sigma \sqrt{(2\pi)}) \exp[-(u-u)^2/2\sigma^2],$$

where σ is the population standard deviation of speed. Therefore,

$$g(\tau) = D/\sigma \tau^2 \sqrt{(2\pi)} \exp[-(D/\tau - D/\tau)^2/2\sigma^2].$$

Letting $s = \sigma / D$,

$$g(\tau) = 1/\tau^2 s \sqrt{(2\pi)} \exp\{[-(1/\tau - u/D)^2]/2s^2\}.$$

Considerating now the two observation points, the number of cars passing the first point is q_1dt in the interval [t, (t + dt)]. Of this flow, the number of cars passing the second observation point will be $q_1(t)g(\tau)dtdT$ in the interval [$(t + \tau)$, $(t + \tau + dT)$].

The total number of cars passing point two in the interval [T, (T + dT)] is $q_2(T)dT = \int_t q_1(t)g(T-t)dtdT$, the integral being overall values of t for which $q_1(t)$ exceeds zero. As the flow pattern will not be represented by a continous curve whose equation is known but by a histogram, it is more convenient to use a corresponding formula for the discrete case. Thus the following expression is employed:

$$q_2(j) = \sum_{i=1}^{n} q_1(i)g(j-i),$$

where i = count of the first observation point,

j = count of the second observation point,

$q_2(j)$ = flow past point 2 during the j-th interval,

$q_1(i)$ = flow past point 1 during the i-th interval,

$g(j-i)$ = probability of a journey time of a duration $(j - i)$intervals, and

n = number of intervals considered (e.g. number of increments in the cycle length).

Putting this formula into words, the flow in the j-th interval at the second observation point equals the sum over all the values of i, of the flow in the i-th interval at the first point multiplied by the probability of a journey time $(j - i)$ intervals.

Pacey found that in general, the diffusion theory fits the observed data fairly well, but that it is slightly inferior to the Lightham's and Whitham's wave theory in extreme traffic conditions. For this reason, Pacey's platoon diffusion theory has not been used in any signal control design procedure.

2.2.3 Robertson's Recurrence Model

Robertson[12] introduced his method of predicting platoon behaviour, in his contribution to the discussion at the Institute of Civil Engineers, Joint Symposium On Area Control Of Road Traffic (London, England) in 1967. His work was carried out at the Road Research Laboratory and is

further reported[13] in the description of the TRANSYT (TRAffic Network StudY Tool) method of coordinating traffic signals. The method is simple to apply and makes use of an empirical recurrence relationship.

This relationship can be illustrated by reference to Figure 2.2. The unit flow in the i-th interval of the initial cyclic flow profile[26] is multiplied by a factor F to give a portion of the unit flow in $(i+\beta T)$ interval in the predicted cyclic flow profile. In addition, $(i+\beta T-1)$ interval of the predicted cyclic flow profile is multiplied by a factor $(1 - F)$ and added to the $(i+\beta T)$ interval of the predicted cycle flow profile. By repeating this process for all flow intervals, the downstream arrival pattern can be derived. In summary, the factor F controls the rate at which a platoon disperses while the effect of introducing βT is to offset the predicted pattern in time by an equal amount, or to ensure that no vehicles arrive at the downstream stopline until βT time intervals have elapsed. Robertson's recurrence model takes the following mathematical form :

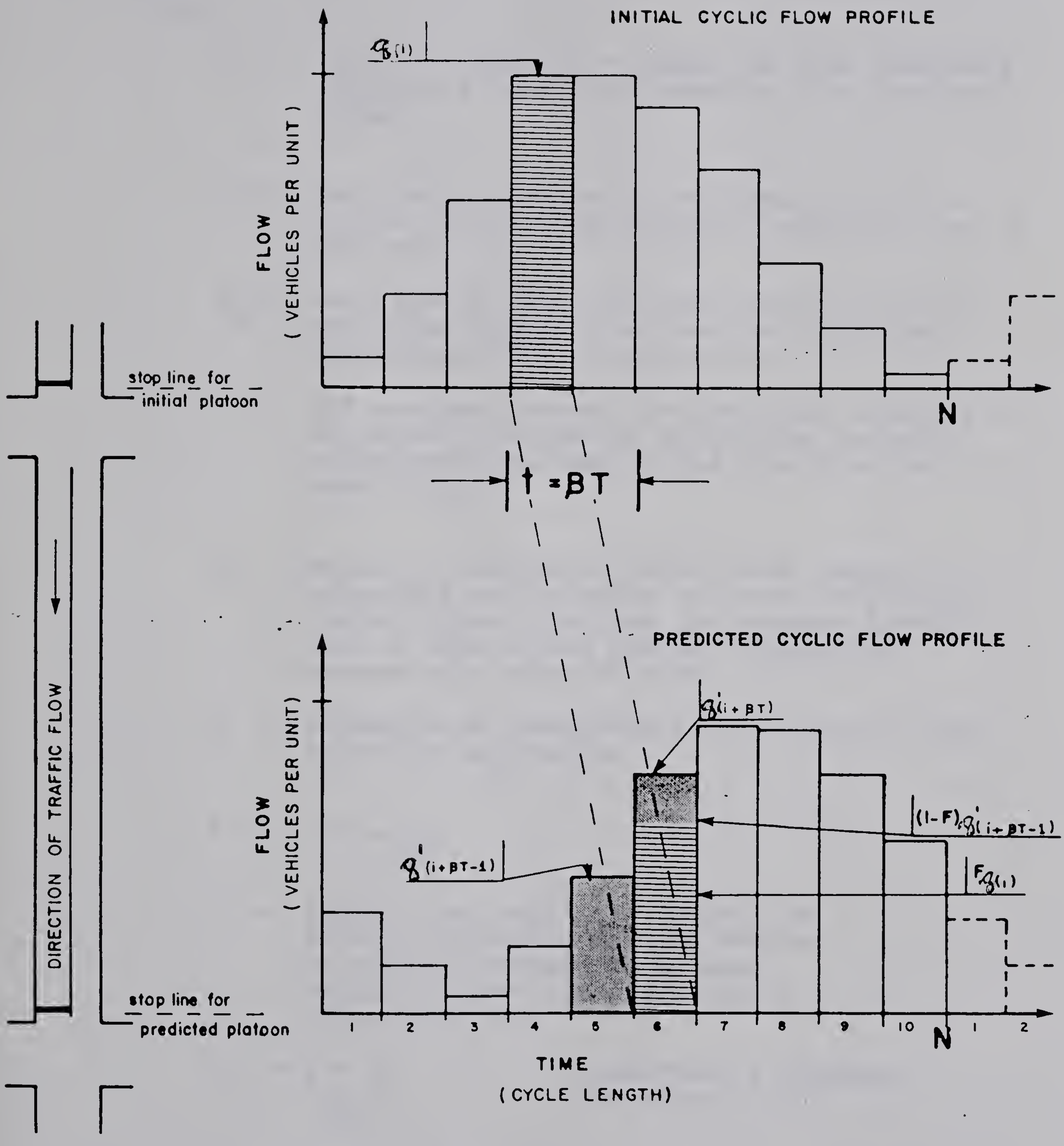


FIGURE 2.2

ROBERTSON'S RECURRENCE MODEL

$$q'_{(i+\beta T)} = Fq_{(i)} + (1 - F)q'_{(i+\beta T-1)} ,$$

where :

i = 1,2,...n; where n = number of time intervals comprising the cycle length at the upstream signal,

q_i = the flow in the i-th time interval of the initial cyclic flow profile (upstream flow in interval i),

q'_i = the flow in the i-th time interval of the predicted cyclic flow profile (predicted downstream flow in interval i),

T = the average journey time over the distance for which the platoon dispersion is being calculated (meaured in the time interval used for q_i),

β = (beta), an empirical travel time factor expressed as the ratio between the platoon leader travel time and the average travel time of the entire platoon (Robertson suggested a value of 0.8),

F = an empirical smoothing factor given by the following expression,

$$F = \frac{1}{1 + \alpha\beta T}$$

α = (alpha), an empirical dispersion factor accounting for the degree of platoon dispersion between signals (Robertson suggested an value of 0.50),

$$K = \frac{\alpha * \beta}{100}$$ (Robertson's Standard K value = 40)

βT = integer value.

For no dispersion of traffic platoons, alpha would assume a value of zero, beta would assume a value of one, and the smoothing factor F would equate to a value of one.

In the limit, as the number of intervals approaches infinity, the predicted cycle flow profile approaches zero, but since the time period is not infinity but rather a cycle length, small errors in the predictive process are introduced. It should also be noted that Robertson's equation requires the flow value in the previous downstream interval to predict the value in the present interval. This information is not available in computing the first downstream interval. Hence Robertson's equation will consistently underestimate the downstream flow rates.

It is important to note that the exponentially smoothing process described above relates to the average platoon behaviour. In reality, individual platoons will vary randomly around the mean. (It should be pointed out that the equation on page 141 of Reference ([12]) has been misprinted.)

Robertson([13]) stated that it would be reasonable to expect that the value of F would be a function of various site factors such as road width, gradient, parking acitiviy, opposing flow level, traffic composition, etc., rather than simply the average journey time.

Although it appears that Robertson devised the method emperically, its undoubted success in signal control design procedures suggests that it must have a sound theoretical basis. Seddon([19]) has shown that Robertson's formula is the

same as that of Pacey, except that Pacey's transformed normal distribution of journey g(j-i) is replaced by a probability function $F(1 - F)^{(j-t-i)}$. This latter function is a geometric distribution which is commonly used for representing the number of failures in a series of trials to the first success. In this case, it is the probability that a vehicle passing the first point in the i-th interval will pass the second point in the j-th interval. Seddon therefore concludes that the success of Robertson's Recurrence Model can be attributed to the approximation of the geometric distribution to the true distribution of journey times.

2.3 Mathematical Extension To Robertson's Recurrence Model

As outlined in Section 2.2.3, Robertson's Recurrence Model for platoon dispersion makes use of the following simple recurrence relationship:

$$q'_{(i+\beta T)} = Fq_{(i)} + (1-F)q'_{(i+\beta t-1)} , \qquad (1)$$

where :

- i = 1,2,...n; where n=number of time intervals comprising the cycle length at the upstream signal,
- q_i = the flow in the i-th time interval of the initial cyclic flow profile (upstream flow in interval i),
- q'_i = the flow in the i-th time interval of the predicted cyclic flow profile (predicted downstream flow in interval i),
- T = the average journey time over the distance for which the platoon dispersion is being calculated (measured in the time intervals used for q_i),
- β = (beta), an empirical travel time factor expressed as the ratio between the platoon leader travel time and the average travel time of the entire platoon (Robertson suggested a value of 0.8),
- F = $\dfrac{1}{1 + \alpha\beta T}$
- α = (alpha), an empirical dispersion factor accounting for the degree of platoon dispersion between signals (Robertson suggested an value of 0.50),
- K = $\dfrac{\alpha*\beta}{100}$ (Robertson's Standard K value=40), and
- βT = integer value.

Robertson's equation requires the flow value in the previous downstream interval [$q'_{(i+\beta T-1)}$] to predict the value in the present interval [$q'_{(i+\beta T)}$]. This information is not available in computing the flow during the first iteration and is assumed to take a zero value. Therefore Equation(1) takes the following form for the first interval only:

$$q'_{(1+\beta T)} = Fq_{(1)} \tag{2}$$

and

$$q'_{(i+\beta T)} = Fq_{(i)} + (1-F)q'_{(i+\beta T-1)} \tag{3}$$

for i=2,3,4,...,n (for all succeeding intervals).

The use of equation (2) underestimates the flow rate during the first interval, and this effect is perpetuated throughout the entire prediction process due to its recursive nature. To provide some insight into this volume error, an expression is derived([27]) for determining the discrepancy between initial and predicted cyclic flow profiles:

$$q'_{(i+\beta T)} = Fq_{(i)} + (1-F)q'_{(i+\beta T-1)} \tag{1}$$

By summing $q'_{(i+\beta T)}$ over the entire cycle which consists of n intervals, Equation (1) can be re-written as follows :

$$\sum_{i=1}^{n} q'_{(i+\beta T)} = F \sum_{i=1}^{n} q_{(i)} + (1-F) \sum_{i=1}^{n} q'_{(i+\beta T-1)} \tag{4}$$

The expansion of the term $\sum_{i=1}^{n} q'_{(i+\beta T-1)}$ is as follows :

$$\sum_{i=1}^{n} q'_{(i+\beta T-1)} = q'_{(\beta T)} + q'_{(1+\beta T)} + q'_{(2+\beta T)} +$$

$$q'_{(3+\beta T)} + \ldots + q'_{(n+\beta T-1)} \; . \tag{5}$$

Adding $q'_{(n+\beta T)} - q'_{(n+\beta T)}$ which equals to a zero value to Equation (5) we obtain the following :

$$\sum_{i=1}^{n} q'_{(i+\beta T-1)} = q'_{(\beta T)} + q'_{(1+\beta T)} + q'_{(2+\beta T)} +$$

$$q'_{(3+\beta T)} + \ldots + q'_{(n+\beta T-1)} +$$

$$q'_{(n+\beta T)} - q'_{(n+\beta T)} \tag{6}$$

$$= q'_{(\beta T)} + \sum_{i=1}^{n} q'_{(i+\beta T)} - q'_{(n+\beta T)} \; . \tag{7}$$

By replacing the equality defined in Equation (7) into Equation (4) we obtain the following :

$$\sum_{i=1}^{n} q'_{(i+\beta T)} = F \sum_{i=1}^{n} q_{(i)} + (1-F)[\, q'_{(\beta T)} +$$

$$\sum_{i=1}^{n} q'_{(i+\beta T)} - q'_{(n+\beta T)} \,]$$

$$= F \sum_{i=1}^{n} q_{(i)} + (1-F)[\, q'_{(\beta T)} - q'_{(n+\beta T)} \,] +$$

$$(1-F) \sum_{i=1}^{n} q'_{(i+\beta T)}$$

$$\sum_{i=1}^{n} q'_{(i+\beta T)} - (1-F) \sum_{i=1}^{n} q'_{(i+\beta T)}$$

$$= F \sum_{i=1}^{n} q_{(i)} + (1-F)[\, q'_{(\beta T)} - q'_{(n+\beta T)} \,]$$

$$F \sum_{i=1}^{n} q'_{(i+\beta T)} = F \sum_{i=1}^{n} q_{(i)} + (1-F)[\, q'_{(\beta T)} - q'_{(n+\beta T)} \,] \quad (8)$$

Dividing by the factor F in Equation (8) we obtain the following :

$$\sum_{i=1}^{n} q'_{(i+\beta T)} = \sum_{i=1}^{n} q_{(i)} + ((1-F)/F)[\, q'_{(\beta T)} - q'_{(n+\beta T)} \,] \quad (9)$$

Under steady state conditions, if it exists, the value of $q'_{(\beta T)}$ and $q'_{(n+\beta T)}$ are identical and therefore the last term in equation(9) is reduced to zero. Under this traffic condition there will be no volume error between the initial cyclic flow profile $[\sum_{i=1}^{n} q_{(i)}]$ and the predicted cyclic flow profile $[\sum_{i=1}^{n} q'_{(i+\beta T)}]$. In practice, however, Robertson's equation assumes that $q'_{(\beta T)}$ has an assumed value of zero, with the value of $q'_{(n+\beta T)}$ being calculated in the final step of the iteration process. This difference in value therefore leads to a finite volume error.

To understand the potential impact of this finite volume error let's consider the feasible values that (1-F)/F and $[\, q'_{(\beta T)} - q'_{(n+\beta T)} \,]$ may assume under realistic traffic flow conditions. For a typical lane lets assume that the smoothing factor F takes on a value of 0.20 and $q'_{(n+\beta T)}$ takes on a value of 1.70. The value of 1.70 is based on a 2 second time slice, saturation flow rate = 1800 vehicles per hour green and 85% degree of saturation. (Saturation flow is defined as the number of passengers car units which can discharge at a stop-line of an "ideal" intersection approach (width 3.0 to 3.5m), moving straight ahead (i.e., no turning movements are involved), with no additional traffic friction (i.e., no bus stops) under the given set of weather conditions, during the given length of the effective green

interval. The concept of saturation flow assumes that after the initial hesitation following the display of green, traffic discharges at a constant rate, until shortly after the beginning of amber, when a sharp drop occurs. Effective green time is therefore defined as the horizontal dimension of the rectangular transformation of the area encompassed by the saturation flow diagrams.) Given this condition the predicted downstream cyclic flow profile is in error by 6.8 vehicles. At a 100 second cycle length (10 seconds intergreen period, 80/20 split between major and minor street) the average volume of traffic per cycle would approach to 36.00 vehicles. However, at 85% degree of saturation, the predicted downstream volume is 36.00 - 5.40 = 30.60 vehicles. The resulting degree of saturation based on predicted volumes (30.60 - 6.80 = 23.80) is calculated to be 66%. The potential impact of a 28.6% error in volume estimate due to the prediction process can best be illustrated by making reference to the following TRANSYT (V5 - V6) expression for random delay:

$$Dr = \frac{(X)^2}{4(1-X)}$$

where: Dr = random delay in units of vehicle hour per hour, and

X = degree of saturation, defined as the ratio of volume to capacity.

Hence for X=0.66 and 0.85 the respective values of Dr are 0.3 and 1.2. The basic conclusion, therefore is that a 28.6% error in volume estimate under high degrees of saturation could increase the random delay component by a factor of four. On the other hand, there is a different set of other traffic conditions in which these types of errors due to the prediction process, can be minimized. Two conditions have been identified. The first condition includes a larger smoothing factor which will result in the value of (1-F)/F approaching zero. The second condition includes very compact platoon in the middle of a long cycle length with no flows on either side of the platoon. The leading edge of this particular downstream platoon would arrive no earlier than $(i+\beta T)$ time units with the cycle length being of sufficient length that the value of $q'_{(i+\beta T)}$ approaches a zero value at the end of the calculation.

Since traffic platoons moving between fixed time signals are cyclic in nature (the period equals the cycle length), and if steady state conditions exist, the flow for an interval will be the same over all cycle lengths. Based on this information an initialization equation is derived.

As was indicated previously, Robertson's equation requires that the flow value in the previous downstream interval [$q'_{(i+\beta T-1)}$] to predict the value in the present interval [$q'_{(i+\beta T)}$]. For the first downstream interval [$q'_{(1+\beta T)}$] the previous downstream interval [$q'_{(1+\beta T-1)}$] is unknown and is therefore assumed to be zero. Applying the

summation of series techniqiue and if steady state conditions exist [27] Robertson's Recurrence Model is expanded to provide an initial estimate for the value of $q'_{(i+\beta T)}$ in the predicted cyclic flow profile. The mathematical proof follows.

Robertson's model is given by the following form:

$$q'_{(i+\beta T)} = F\, q_i + (1-F)\, q'_{(i+\beta T-1)} \quad . \qquad (1)$$

Redefine equation (1) by looking at the previous interval.

$$q'_{(i+\beta T-1)} = F\, q_{(i-1)} + (1-F)\, q'_{(i+\beta T-2)} \qquad (10)$$

Substitute equation (10) into equation (1). Equation (1) takes the following form:

$$q'_{(i+\beta T)} = F\, q_i + (1-F)[F\, q_{(i-1)} + (1-F)\, q'_{(i+\beta T-2)}]$$

$$= F\, q_i + F(1-F)\, q_{(i-1)} + (1-F)^2\, q'_{(i+\beta T-2)} \quad . \qquad (11)$$

Redefine equation (10) by looking at the previous interval.

$$q'_{(i+\beta T-2)} = F\, q'_{(i+\beta T-2)} + (1-F)\, q'_{(i+\beta T-3)} \qquad (12)$$

Substitute equation (12) into equation (11). Equation (11) takes the following form:

$$q'_{(i+\beta T)} = F\, q_i + F(1-F)\, q_{(i-1)} + (1-F)^2[F\, q_{(i-2)} + (1-F)\, q'_{(i+\beta T-3)}]$$

$$= F\, q_{(i)} + F(1-F)\, q_{(i-1)} + F(1-F)^2\, q_{(i-2)} +$$

$$(1-F)^3 \, q'_{(i+\beta T-3)} \tag{13}$$

Re-define equation (12) by looking at the previous interval.

$$q'_{(i+\beta T-3)} = F \, q_{(i-3)} + (1-F) \, q'_{(i+\beta T-4)} \tag{14}$$

Substitite equation (14) into equation (13). Equation (13) takes the following form:

$$q'_{(i+\beta T)} = F \, q_{(i)} + F(1-F) \, q_{(i-1)} + F(1-F)^2 \, q_{(i-2)} +$$

$$(1-F)^3 [F \, q'_{(i-3)} + (1-F) \, q'_{(i+\beta T-4)}]$$

$$= F \, q_i + F(1-F) \, q_{(i-1)} + F(1-F)^2 \, q_{(i-2)} +$$

$$F(1-F)^3 \, q_{(i-3)} + (1-F)^4 \, q'_{(i+\beta T-4)} \tag{15}$$

It follows that $q'_{(i+\beta T)}$ can be structured in the following general form:

$$q'_{(i+\beta T)} = F \, q_i + F(1-F)^1 \, q_{(i-1)} + F(1-F)^2 \, q_{(i-2)}$$

$$+ \ldots + F(1-F)^{n-1} \, q_{(i-n+1)} + (1-F)^n \, q'_{(i+\beta T-n)} \tag{16}$$

More specifically, equation (16) translates to the following:

$$q'_{(i+\beta T)} = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(i-j)} + (1-F)^n \, q'_{(i+\beta T-n)} \tag{17}$$

Equation (17) and equation (1) are identical. If the number of time slices within a cycle length is n and we assume that the value from the predicted cyclic flow profile is the same at the beginning of each cycle then:

$$q'_{(i+\beta T-n)} = q'_{(i+\beta T)} = q'_{(i+\beta T+n)}$$

$$= q'_{(i+\beta T+2n)} \quad \text{etc.} \tag{18}$$

This equality follows from the key assumuption that steady state conditions prevail. For interval i=1 equation (17) and equation (18) become the following, respectively:

$$q'_{(1+\beta T)} = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(1-j)} + (1-F)^n \, q'_{(1+\beta T-n)} \tag{19}$$

$$q'_{(1+\beta T-n)} = q'_{(1+\beta T)} = q'_{(1+\beta T+n)} = q'_{(1+\beta T+2n)}$$
etc. (20)

Substitute $q'_{(1+\beta T-n)}$ in equation (19) with $q'_{(i+\beta T)}$ as defined by the equality in equation (20).

$$q'_{(1+\beta T)} = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(1-j)} + (1-F)^n \, q'_{(1+\beta T)} \tag{21}$$

Therefore :

$$q'_{(1+\beta T)} - [(1-F)^n \, q'_{(1+\beta T)}] = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(1-j)} \tag{22}$$

$$q'_{(1+\beta T)} \, (1-(1-F)^n) = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(1-j)} \tag{23}$$

$$q'_{(1+\beta T)} = \sum_{j=0}^{n-1} F(1-F)^j \, q_{(1-j)} / [1-(1-F)^n] \tag{24}$$

Equation (24) can be used not only for initial interval prediction but also for computing all intervals simply by substituting the indices $(1+\beta T)$ and $(1-j)$ by $(i+\beta T)$ and $(i-j)$ respectively. However, this equation is more cumbersome to use than Robertson's equation, and after the first interval would not improve the model accuracy. The incorporation of this equation into the Robertson Model will in subsequent sections of this thesis be referred to as the Modified Robertson Recurrence Model. It is anticipated that this improvement to the model will significantly affect the calibration process. Since the upstream and downstream cyclic flow profiles will have the same volume, the result will be better predictions with respect to total delay (random delay + uniform delay), number of vehicle stops and performance index.

3. DATA COLLECTION EFFORTS

3.1 Data Collection Techniques

The collection of real-time, traffic platoon data was achieved with the aid of a Micro-computer Cyclic Flow Profile Monitor([28]). Travel time information was derived by collecting spot speed with the aid of a radar speed gun. Technical details of each of these two tools are discussed below.

3.1.1 Platoon Data

Research at the University of Alberta is currently underway to develop a Micro-computer Cyclic Flow Profile Monitor. The specific objective of this research, which is under the auspices of both the Department of Electrical Engineering and the Department of Civil Engineering, at the University of Alberta, is as follows:

> " To Design And Build A Micro-computer Based Local Device To Monitor Cyclic Flow Profiles Within The Context Of An Incident Detection System. "

With the support of the City of Edmonton, Transportation Department (formerly the Engineering Department), a basic system has been designed and built to permit numerous users to collect several different types of traffic related information.

The system configuration of this device makes use of an eight bit micropocessor chip. Erasable Programable Read Only

Memory (EPROM) provides program storage, while Random Access Memory (RAM) is used for data collection purposes. All information collected in RAM is dumped onto a sixty (60) minute cassette tape, via a standard tape recorder. Once on tape, information is then transmitted to the University of Alberta Computing Centre from the City of Edmonton Traffic Control Centre. Figure 3.1 conceptually illustrates a block diagram of the system.

The hardware of the system is described below and is illustrated conceptually in Figure 3.2.

1. An A.C. Transformer is used to convert 12 volts D.C. power from a standard car battery to 120 Volts A.C. power to energize the microprocessor.

2. The entry of the operating system commands for the microprocessor itself is via a standard Keyboard Panel. The keyboard is used initially to start the data collection routine, with the basic input being the cycle time of the upstream signal and a sequence set of commands to set up the work space. A reset button on top of the microprocessor terminates the data collection effort. The keyboard is also used to issue a set of commands to store data onto a standard cassette recorder.

3. The interface card, ribbon cable, and connector box along with the cable wires form the system from which the collection of field data is transmitted to the microprocssor.

4. The length of the cable wire essentially constrain the maximum possible distances between observations points. Each survey location (shown as Location #1, #2 and #3) consists of two channels for recording platoon data. Typically Location #1 uses channels 0 and 1, Location #2 uses channels 2 and 3 and Location #3 uses channels 4 and 5. For consistency, the lower channel at each location usually represents the curb lane of the roadway section.

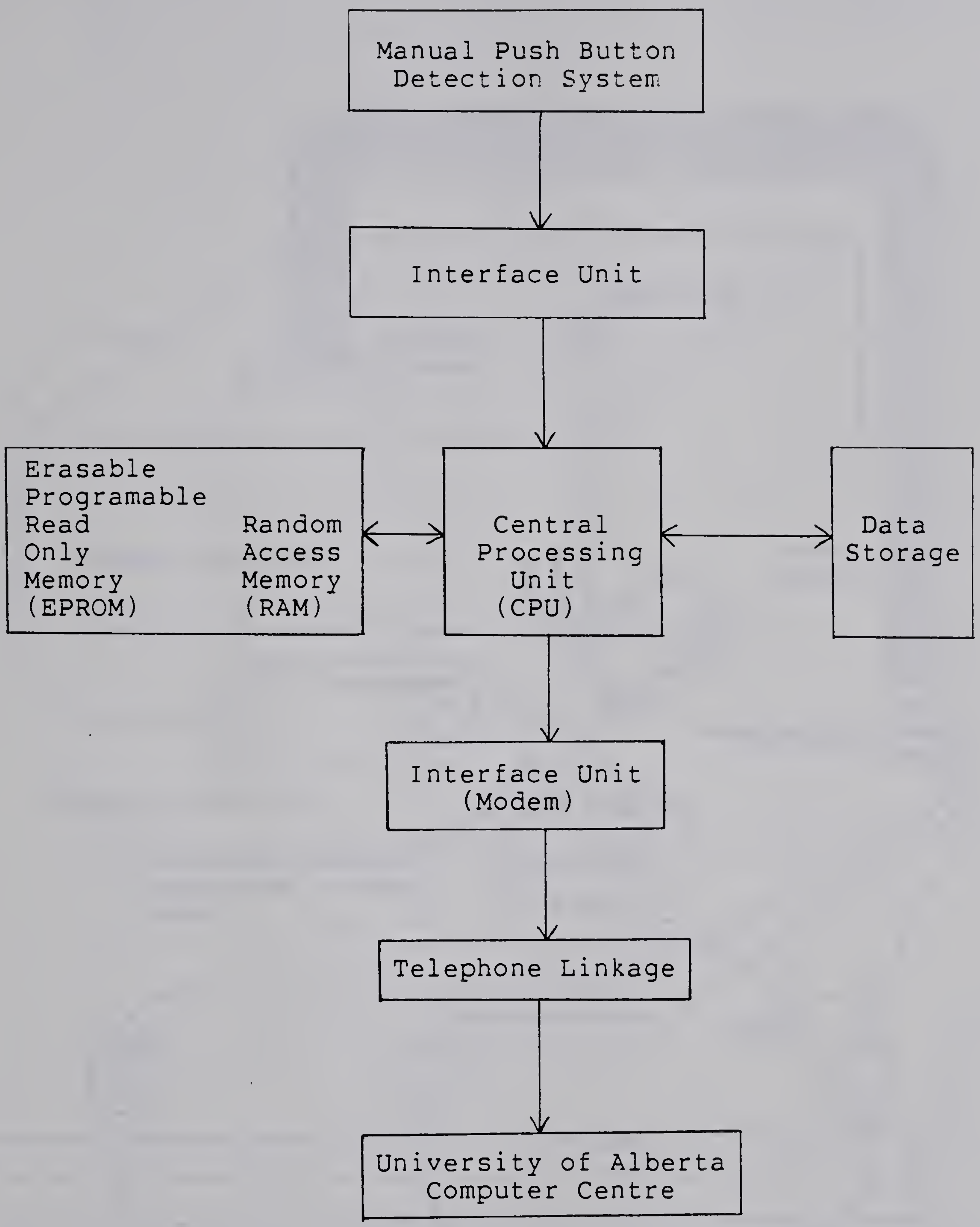


FIGURE 3.1

BLOCK DIAGRAM OF THE
MICRO-COMPUTER CYCLIC FLOW PROFILE MONITOR

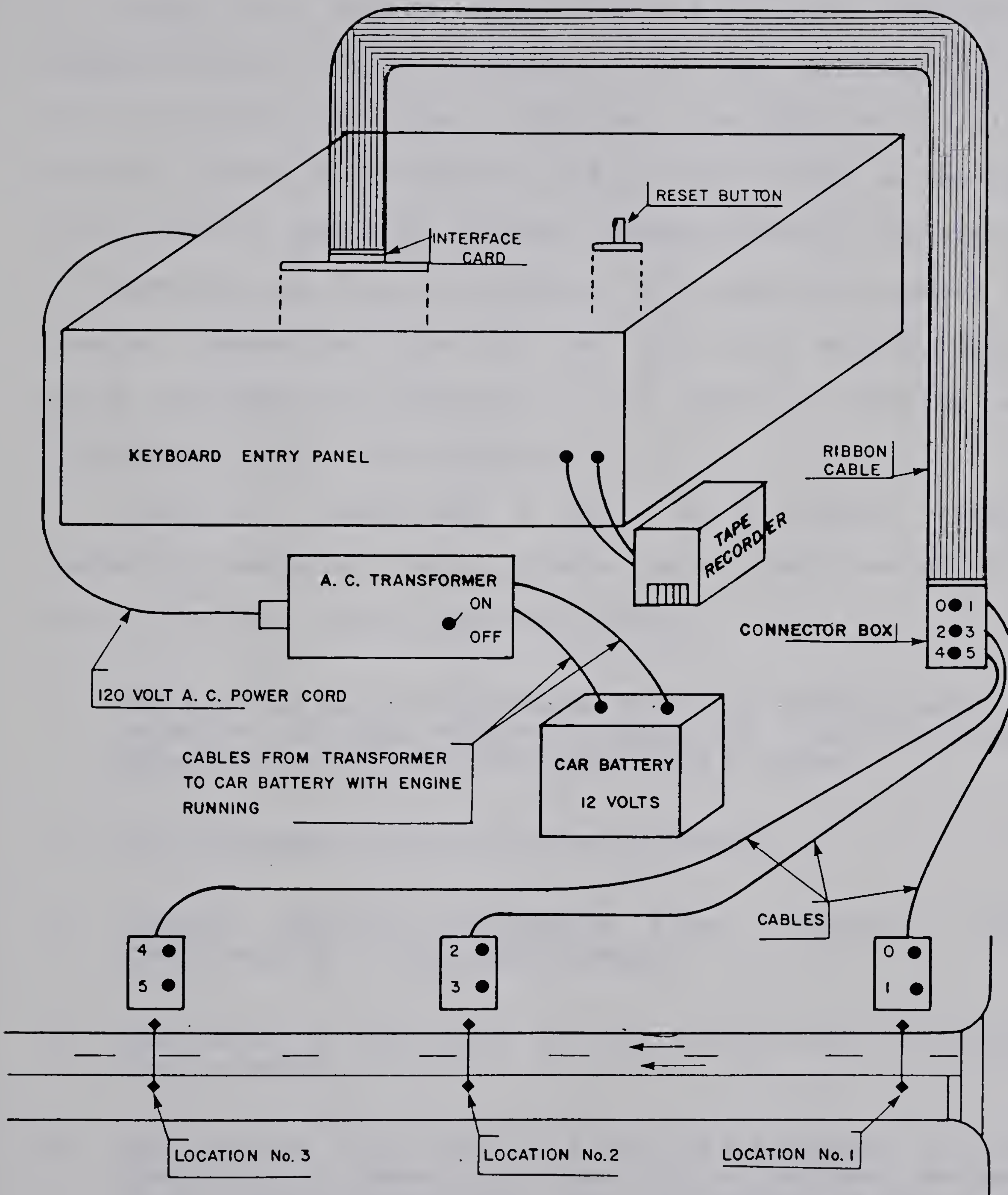


FIGURE 3.2

HARDWARE USED IN THE FIELD
FOR COLLECTING REAL-TIME TRAFFIC PLATOON DATA

5. The collection of the data requires a surveyor at each location. Basic instructions are to press the respective channel number button(s) (event recorder) each time a vehicle passes the survey location.

After the survey, the collected data was stored on a cassette tape, and the information was then transmitted to the University of Alberta Computer from Edmonton's Traffic Control Centre. To translate this raw data into a meaning full format a computing program (BRAD#+*PLOTLIB) was written to summarize the data in a tabular and graphical manner. The tabular summarizes are in the form of an arrival matrix while the graphical plots are in the form of average and individual cyclic flow profiles.

Table 3.1 summarizes a part of a typical tabular summary of measured traffic platoon data. Characteristics of the arrival matrix are described below:

1. The arrival table is prefaced with four header cards to organize the data base with respect to location, survey date/time, run identifier and channel number.

2. Rows represent cycles of data collected.

3. Columns represent successive time intervals (user specified) up to the cycle length.

4. Each entry is the number of vehicles arriving in a given time interval.

5. The observed total number of vehicles recorded in each time slice is summarized along with an unbiased estimate of the standard deviation (The information is the basic input into two computer programs which will be discussed in Chapter 4).

6. The totals for each time interval are also converted

TABLE 3.1

TYPICAL ARRIVAL MATRIX

TABULAR SUMMARY OF MEASURED TRAFFIC PLATOON DATA

LOCATION : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : THURSDAY , FEBRUARY 4 , 1982 ; 7:30 A.M.
RUN IDENTIFIER : WINTER CONDITIONS , EXHAUST FUMES IN THE AIR , ICY PAVEMENT
CHANNEL NUMBER :0 BOTH LANES (CHANNEL 0+1) 80 FEET DOWNSTREAM OF STOPBAR

OBSERVED ARRIVALS BY TIME INTERVAL DURING EACH CYCLE

TIME INTERVAL WITHIN EACH CYCLE (SECONDS)

CYCLE	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	24-26	26-28	28-30	30-32	32-34	34-36	36-38
1	0	1	1	0	2	3	3	1	1	3	2	2	1	3	0	1	0	1	2
2	0	1	0	2	1	3	1	2	3	2	2	2	2	1	1	2	1	2	3
3	0	0	0	2	1	2	2	3	1	3	2	2	1	1	2	2	2	1	1
4	0	0	0	2	2	3	1	3	1	3	1	2	2	2	3	2	2	2	2
5	0	0	0	2	2	2	2	3	2	2	1	2	1	1	3	3	2	2	1
6	0	0	0	1	2	2	2	2	1	2	2	2	1	2	3	2	2	1	1
7	1	0	1	0	1	2	2	2	2	2	3	1	2	2	2	2	2	2	2
8	0	0	0	2	1	2	3	2	2	1	3	2	2	3	1	3	2	2	2
9	0	0	0	1	3	2	3	1	3	3	1	2	2	2	1	1	1	0	2
10	0	1	1	1	2	2	3	2	2	2	2	1	1	2	2	1	2	1	1
11	0	2	0	1	1	1	2	2	1	1	3	3	2	1	2	2	2	2	1
12	0	1	0	1	2	3	2	1	1	2	2	3	2	2	2	0	1	2	2
13	0	1	0	2	2	2	2	2	2	3	3	2	2	0	2	2	1	1	2
14	0	2	0	1	1	2	1	2	1	1	1	1	1	1	1	1	0	0	0
15	2	0	0	1	2	1	1	3	3	2	2	2	2	2	2	2	1	3	1
16	0	0	0	1	2	1	2	1	2	2	1	1	1	1	1	1	0	0	0
17	1	1	1	0	2	1	3	2	2	2	1	3	3	2	2	2	1	2	2
18	1	0	1	1	3	1	2	2	1	2	2	2	1	2	2	1	2	2	2
19	1	0	0	1	1	1	1	2	1	1	1	0	1	1	1	1	1	1	2
20	2	0	1	1	2	2	2	3	2	2	1	2	2	2	3	1	1	2	2
21	0	0	0	2	1	2	2	2	2	3	3	2	2	1	1	3	2	1	0
TOTAL	8	10	6	25	36	40	42	43	36	44	39	39	34	34	37	35	28	30	31
MEAN	0.38	0.48	0.29	1.19	1.71	1.90	2.00	2.05	1.71	2.10	1.86	1.86	1.62	1.62	1.76	1.67	1.33	1.43	1.48
S. DEV	0.67	0.68	0.46	0.68	0.64	0.70	0.71	0.67	0.72	0.70	0.79	0.73	0.59	0.74	0.83	0.80	0.73	0.81	0.81
FLOW RATE (VEHICLES PER HOUR GREEN)																			
MEAN	343	429	257	1071	1543	1714	1800	1843	1543	1886	1671	1671	1457	1457	1586	1500	1200	1286	1329
S. DEV	602	612	417	612	579	630	636	602	645	630	714	654	531	666	748	716	657	730	732

into an average one lane flow rate (vehicles per hour green) along with an unbiased estimate of the standard deviation.

The graphical output from this program includes the following:

1. A plot of the average lane flow rates with an asterisk (*) marker to illustrate the degree of variability (+ or - one standard deviation) in the data base. A typical graphical summary of the arrival matrix is shown in Figure 3.3

2. A plot(s) for a specified cycle(s) to observe the actual lane flow rates for that cycle in comparision to the average lane flow rates for all cycles collected. The average lane flow rates for all cycles collected is indicated with an asterisk (*) marker. A detail graphical summary of the arrival matrix is shown in Figure 3.4

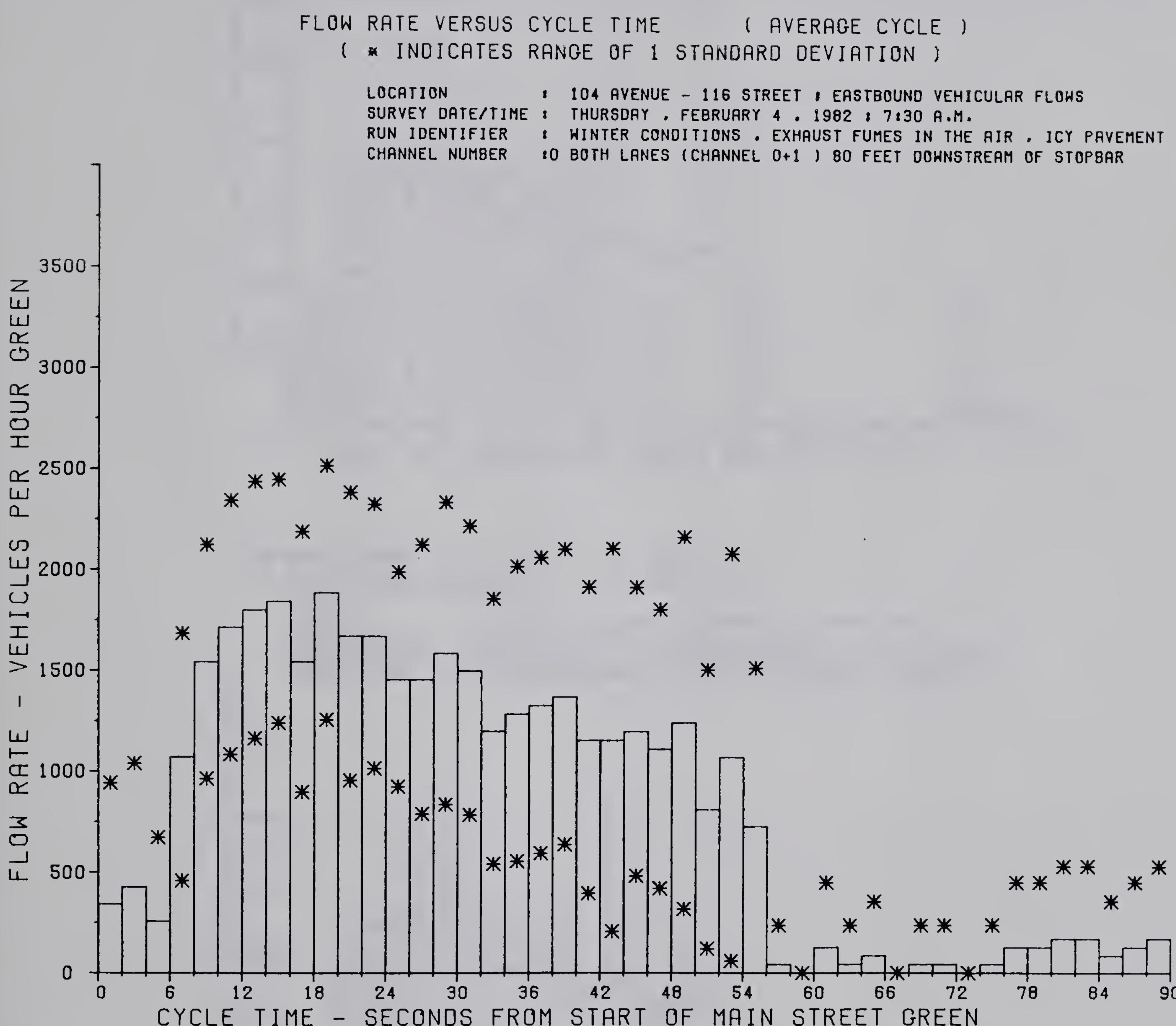
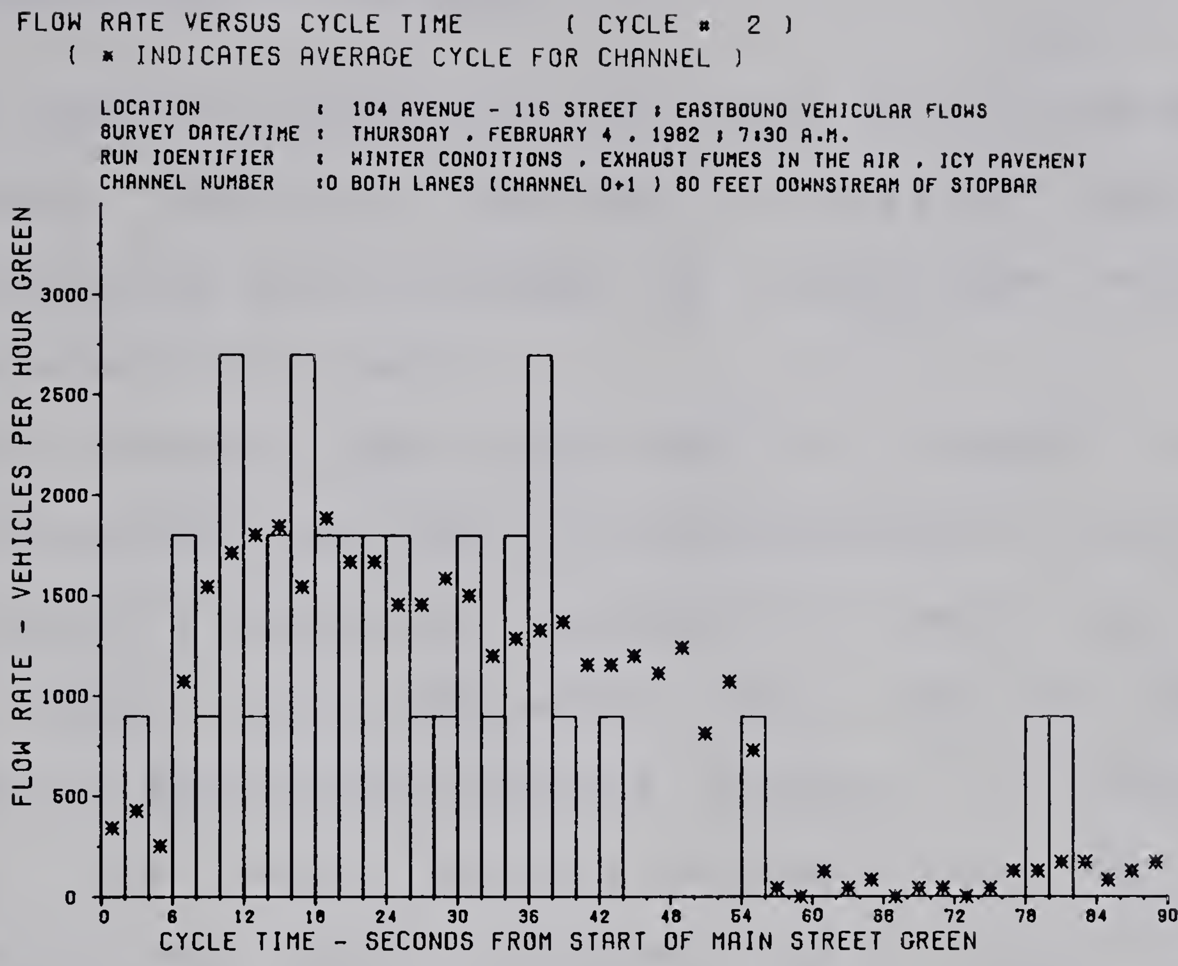


FIGURE 3.3

TYPICAL GRAPHICAL SUMMARY OF THE ARRIVAL MATRIX



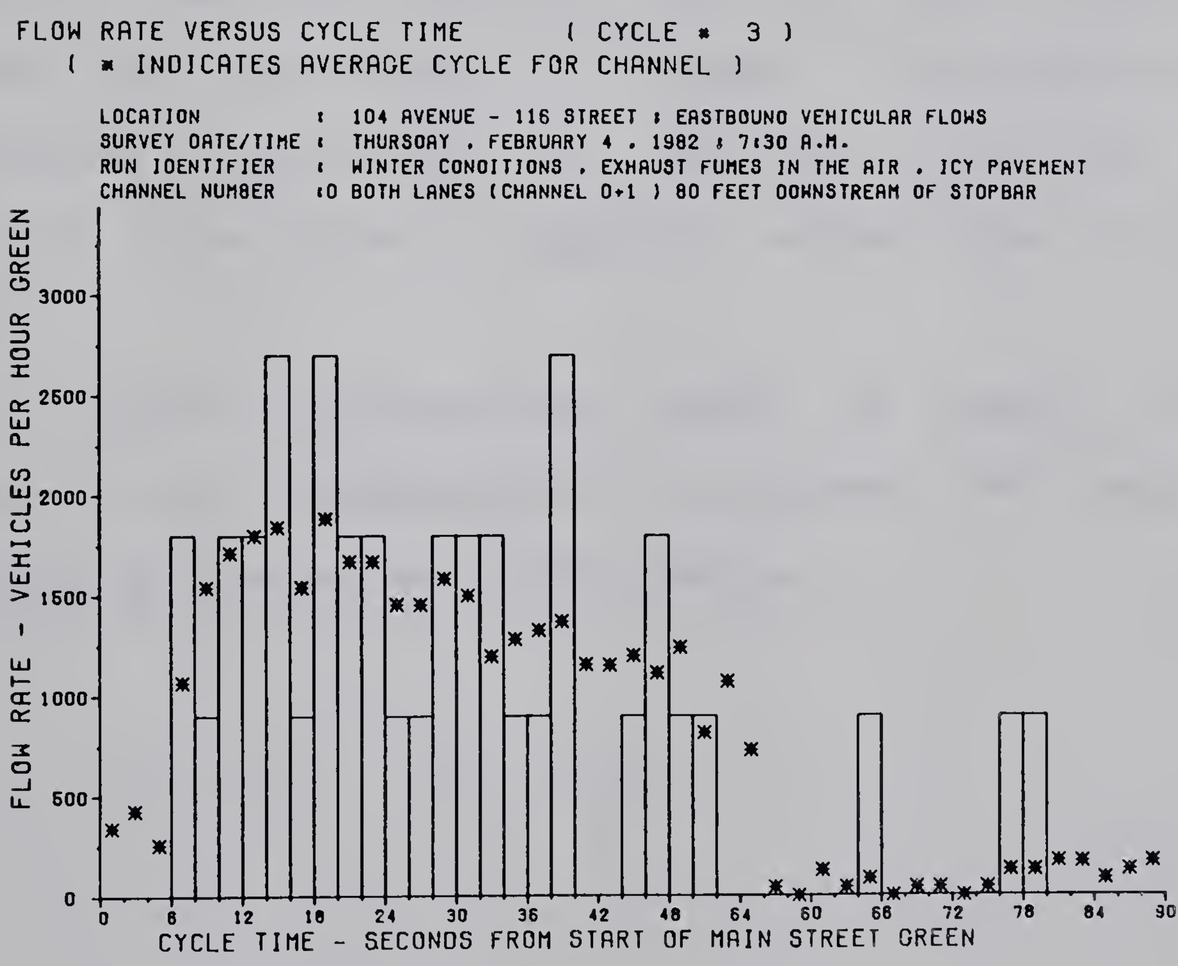


FIGURE 3.4

DETAIL GRAPHICAL SUMMARY OF THE ARRIVAL MATRIX

3.1.2 Speed/Travel Time Data

To obtain an estimate of the actual travel time between two survey locations, the data collected with spot speed survey technique was translated to travel time with the uniform acceleration model.

The standard radar gun which is based on the Doppler-principle was used and calibrated prior to each spot speed survey. A statistical analysis of each spot speed survey date was undertaken using the STPK Package (Interactive STatistical Graphics PacKage [34]. Principal outputs from this analysis included the mean speed (kilometres per hour), variance, and a fitted normal probability density funtion. For all spot speed data collected and analyzed, a Normal Distribution was statistically significant at the 95% confidence level. Figure 3.5 illustrates a typical output from the STPK analysis.

The uniform acceleration model is based on the assumption that acceleration is constant. This results in the following fundamental relationships :

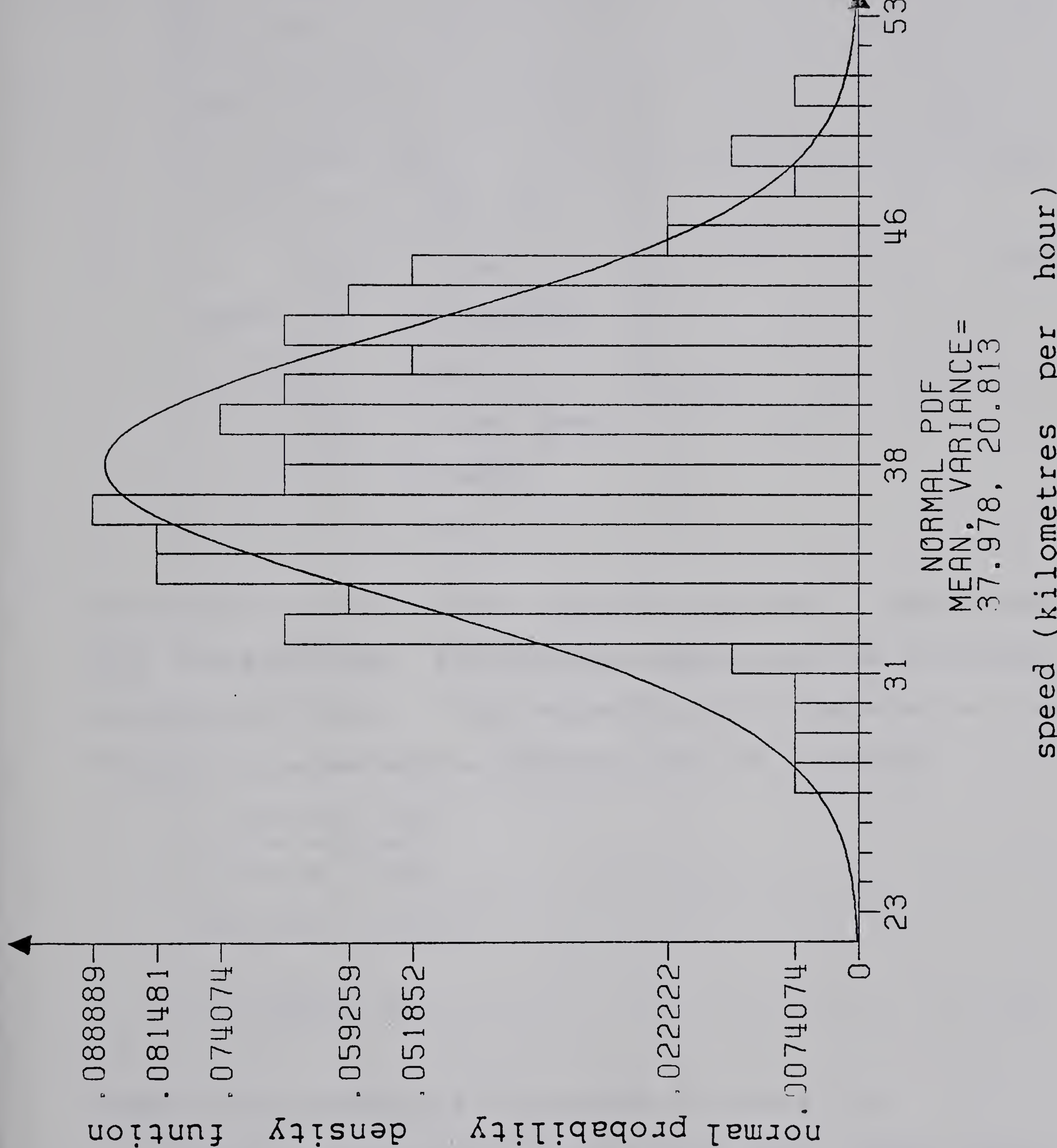


FIGURE 3.5

STPK STATISTICAL ANALYSIS OF TYPICAL SPOT SPEED SURVEY DATA

$$\frac{dv}{dt} = a \tag{1}$$

$$\int_{v_o}^{v} dv = \int_0^t a\, dt$$

$$v = v_o + at \tag{2}$$

$$\int_0^x dx = \int_0^t (v_o + at)\, dt$$

$$x = v_o t + 1/2\, at^2 \tag{3}$$

where a = acceleration

v = speed

v_o = initial speed

x = distance

t = time

Equation (1) to (3) are the acceleration-time, speed-time, and distance-time relationships respectively for uniformly accelerated motion. The expression for distance as a function of speed may be obtained from (1) as follows :

$$a = \frac{dv}{dx}\frac{dx}{dt} = \frac{dv}{dx} v$$

$$\int_0^x a\, dx = \int v\, dv$$

$$x = \frac{1}{2a}(v^2 - v_0^2) \tag{4}$$

These relationships are illustrated in Figure 3.6

Based on several spot speed survey mean speeds along the roadway section, and the uniform acceleration model, an estimate of the travel time between two survey locations was determined.

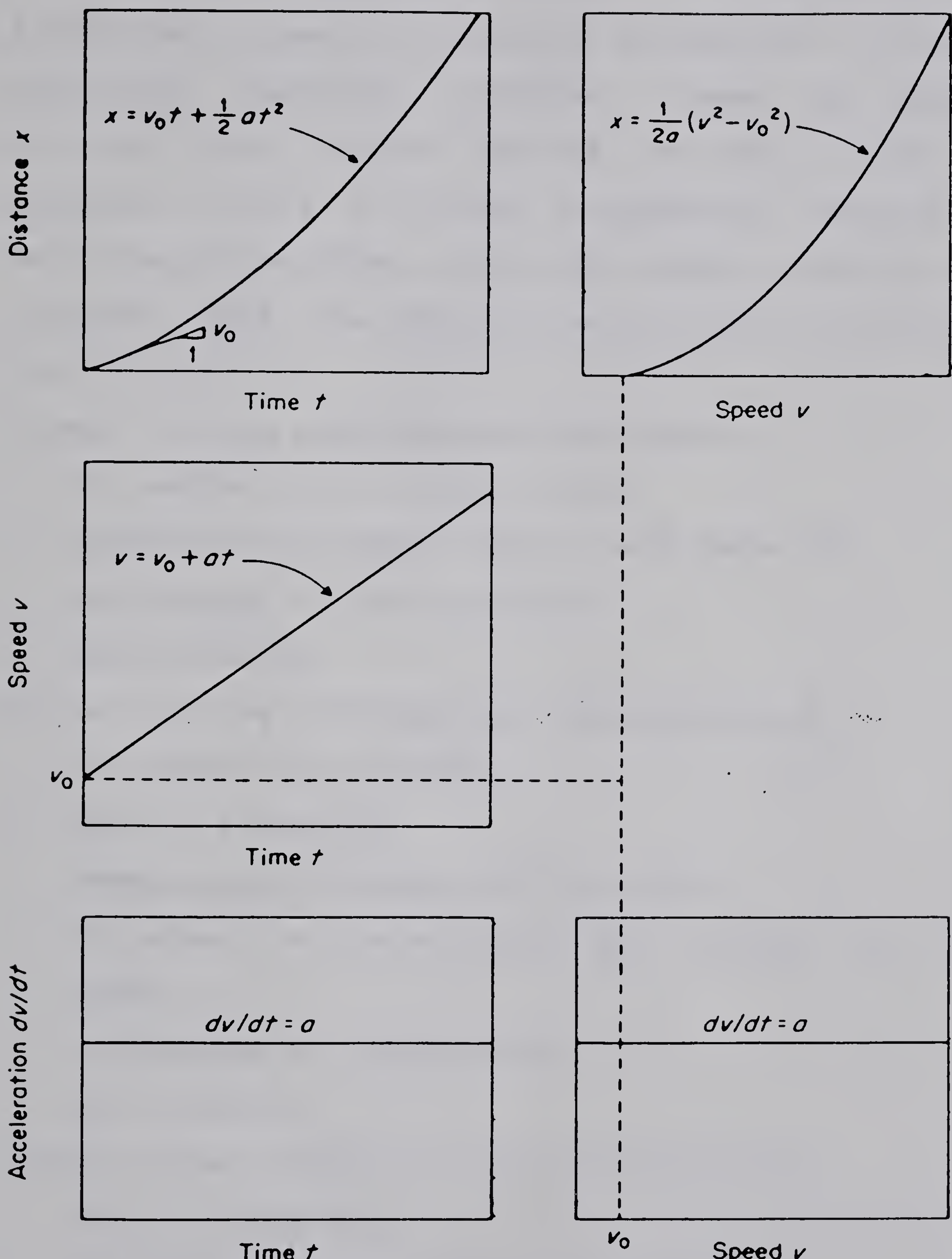


FIGURE 3.6

UNIFORM ACCELERATION MODEL

3.2 The Survey Sites And Their Characteristics

Table 3.2 provides an overview of the survey sites during the A.M. peak hour from the perspective of weather conditions, dispersion distance, number of platoons collected, total volume observed, and the degree of saturation (ratio of volume to capacity) observed. The definition of the three states of weather conditions is consistent with the research done in Alberta on saturation flow ([35]).

"Summer" driving conditions were defined as :

- dry weather (i.e. no rain or snow)
- temperature form about -10°C to well above 0°C
- dry pavement (no snow residues)
- good visibility

"Winter" driving conditions were characterized as :

- dry weather (no snowfall)
- clear or cloudy sky
- temperatures from about -10°C to -30°C
- dry pavement with some packed snow residues but well sanded
- no snowbanks at intersections
- good visiblity

"Severe winter" conditions were characterized as :

- clear or cloudy sky
- very low temperature (lower than -30°C)
- visibility obstructed by exhaust fumes

or by:

TABLE 3.2

OVERVIEW OF THE SURVEY SITES DURING THE A.M. PEAK HOUR

Location	Weather Conditions	Dispersion Distance (meters)	Number Of Platoons Collected	Total Volume (vehicles)	Degree Of Saturation (%)
104 Av. - 116 St.	Severe Winter	106	21	860	92
104 Av. - 116 St.	Winter	94	28	1248	88
104 Av. - 116 St. (Week No. 1)	Summer	106	27	1318	96
104 Av. - 116 St. (Week No. 2)	Summer	106	27	1260	92
111 Av. - 116 St.	Summer	106	25	386	44
102 A Av. - 97 St.	Summer	84	23	157	29

- heavy snowfall or shortly after, before snow has been cleared
- temperatures below 0°C.

As can be seen, the definitions do not provide for a clear seasonal distinction. "Summer" driving conditions may occur during the winter season while "winter" or "severe winter" conditions may be experienced in the spring or fall. Generally, however, the definitions characterized typical long lasting "summer" and "winter" contitions.

The dispersion distance generally varies between 84 - 106 metres, a distance which is generally representative of the majority of roadway links in the CBD, as illustrated in Figure 3.7. The number of platoons collected reflects on average 30 - 45 minutes of field observations per survey site. These observations translated to a volume of 157 to 1248 vehicles with degrees of saturation varying from 29 - 92 %. In summary, more than 150 platoons of data were collected and 5200 individual vehicles were surveyed during these surveys.

3.2.1 104 Avenue - 116 Street (Severe Winter Conditions)

1. Organization of data base for future reference : LOCATION: 104 Avenue -116 Street; Eastbound Vehicular flows. SURVEY DATA/TIME: Thursday, February 4,1981; 7:30 A.M., RUN IDENTIFIER: Severe Winter Conditions, Exhaust Fumes In The Air, Icy Pavement, CHANNEL NUMBERS: Locations #1 (Channel 0 + 1 Combined, 24 Metres (80 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 130 Metres (426 Feet)

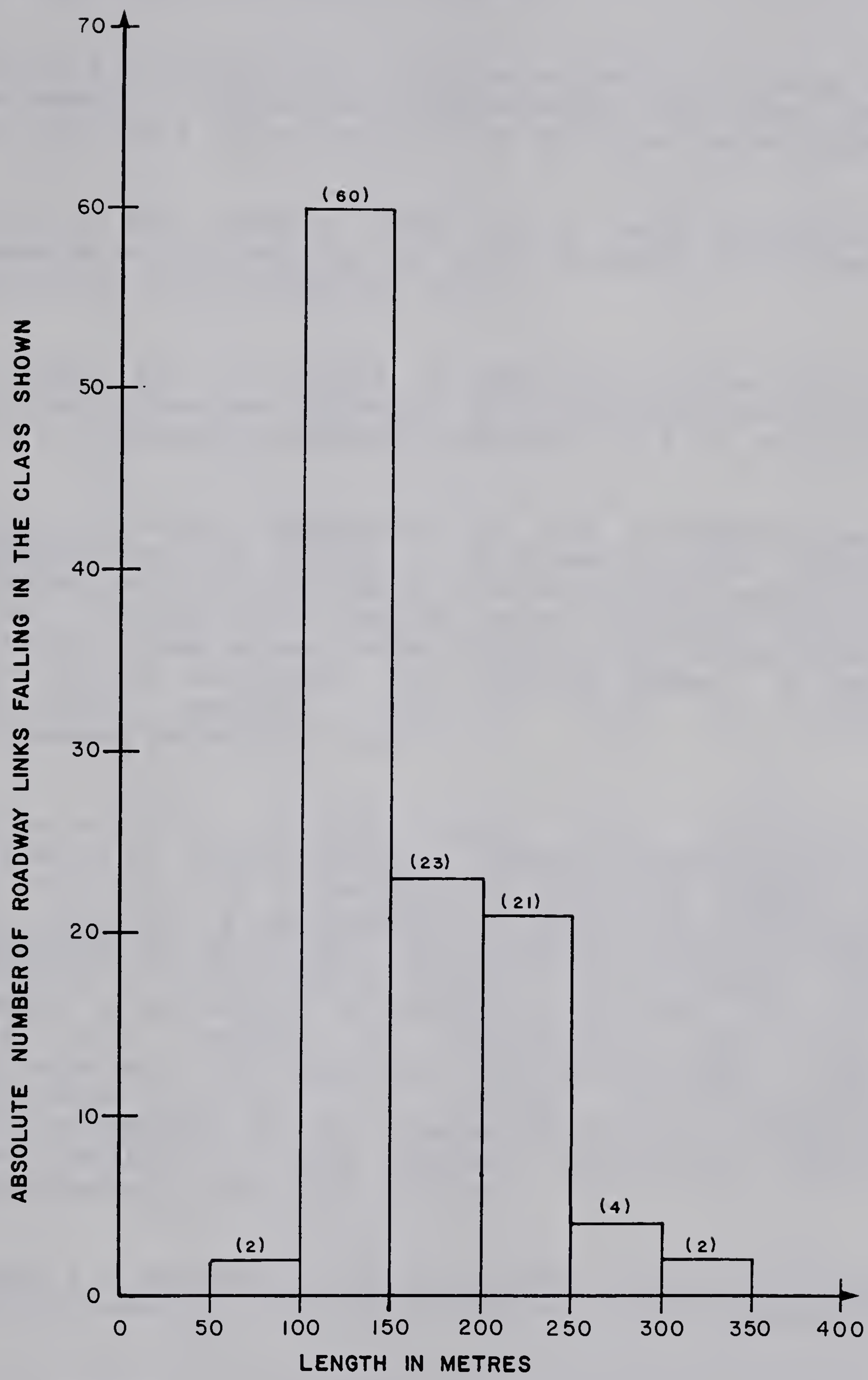
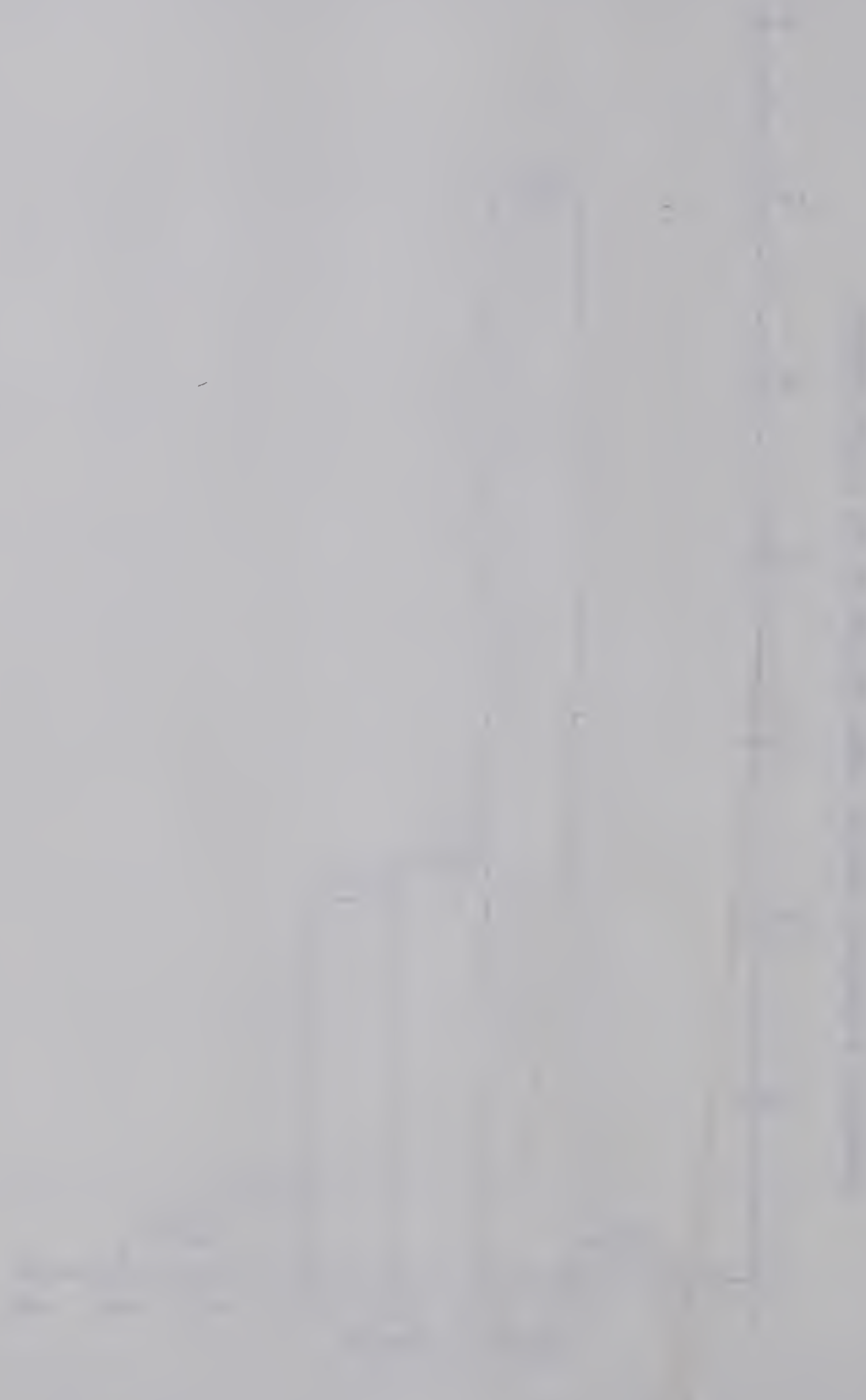


FIGURE 3.7

HISTOGRAM OF THE LENGTH IN METRES BETWEEN ALL SIGNALIZED INTERSECTIONS IN THE CENTRAL BUSINESS DISTRICT OF EDMONTON, ALBERTA



Downstream Of The Signal Stopbar).

2. Figure 3.8 illustrates a scaled schematic drawing of the 104 Avenue - 116 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a distance of 106 Metres) is 14.04 seconds, 24.64 feet per second or 16.80 miles per hour.

4. A total of 21 cycles of data were collected with an average volume per cycle of 40.95 vehicles for two lanes with an unbiased standard deviation of 7.74 vehicles.

5. Signal timing information at the upstream signalized intersection (104 Avenue - 116 Street); cycle length=90 seconds, main street green (104 Avenue)=50 seconds, main street intergreen (3 seconds amber, 2 seconds all-red)=5 seconds, cross street green (116 street)=30 seconds, and cross street intergreen (3 seconds amber, 2 seconds all-red)=5 seconds. Interval size for analysis purposes=2 seconds=1 step.

6. Characteristics of the survey site includes the following : major arterial roadway system leading into CBD, a 92% degree of saturation (<u>at all survey sites this degree of saturation is implied for the upstream signalized intersection</u>), no parking/stopping along the curb, very little side friction (no driveway), passing freedom to manoeuvre is seriously limited due to the very high degree of saturation, bus volume is approximately four buses per hour with a dwell time in the range of 10 - 15 seconds, and the movement of traffic platoons can be characterized as unrestricted (the location of the nearest downstream signal was approximately nine city blocks).

7. Figure 3.9 presents graphically the survey information for 104 Avenue - 116 Street (Severe Winter Conditions).

3.2.2 104 Avenue - 116 Street (Average Winter Conditions, Further Downstream)

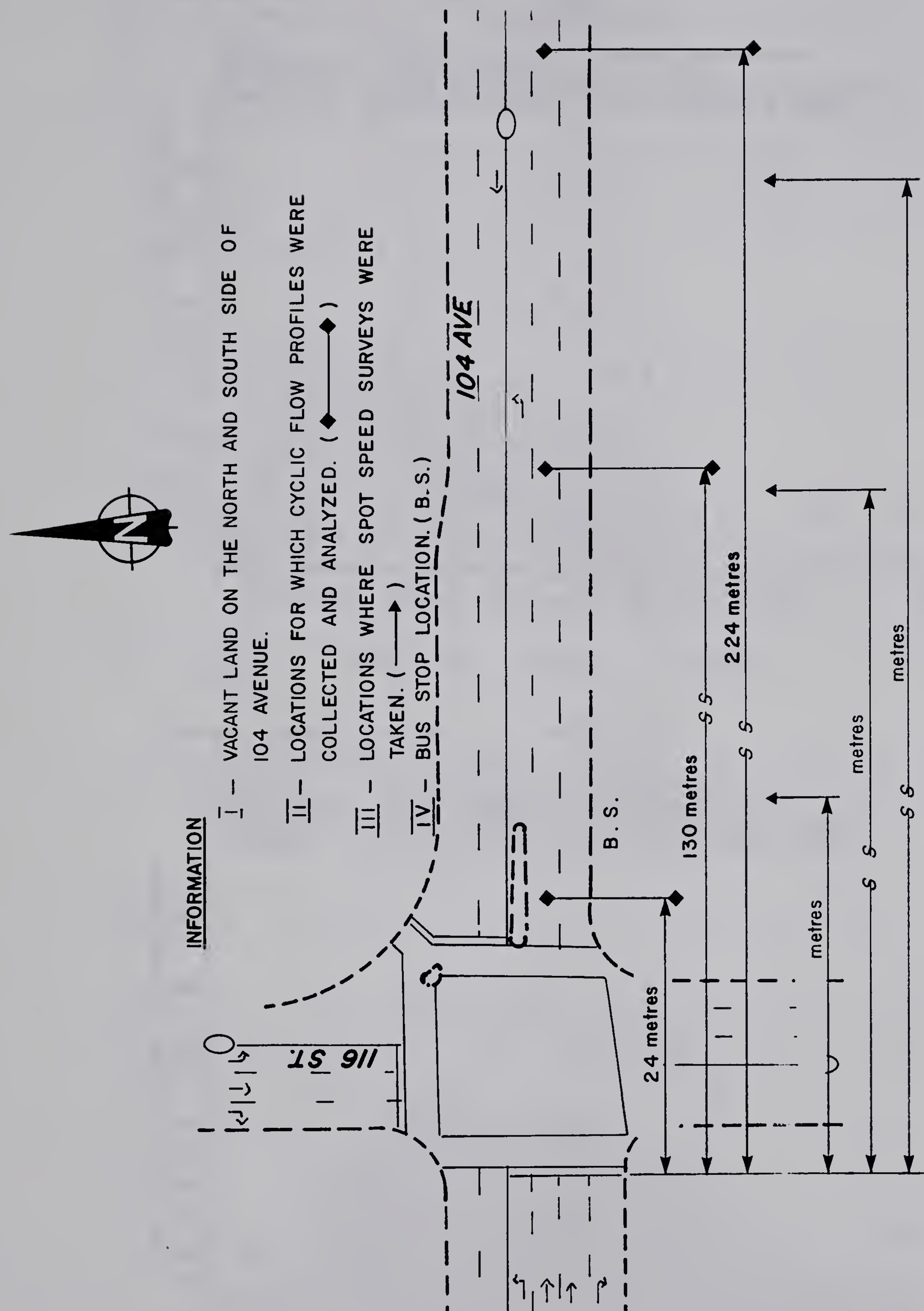
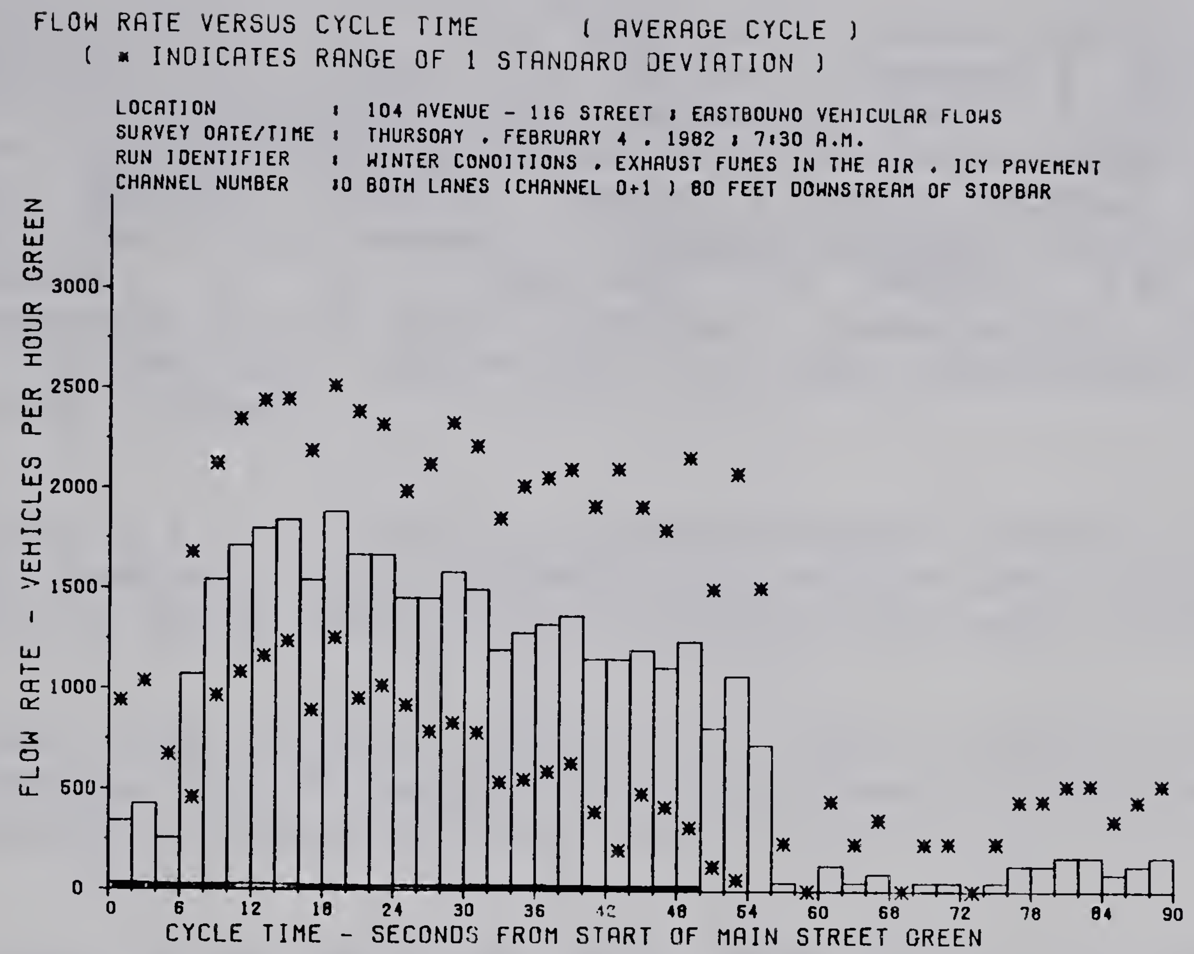


FIGURE 3.8

A SCALED SCHEMATIC DRAWING OF THE 104 AVENUE - 116 STREET INTERSECTION AND THE ROADWAY SECTION WHERE CYCLIC FLOW PROFILES WERE COLLECTED



TRAVEL TIME = 14.04 SECONDS

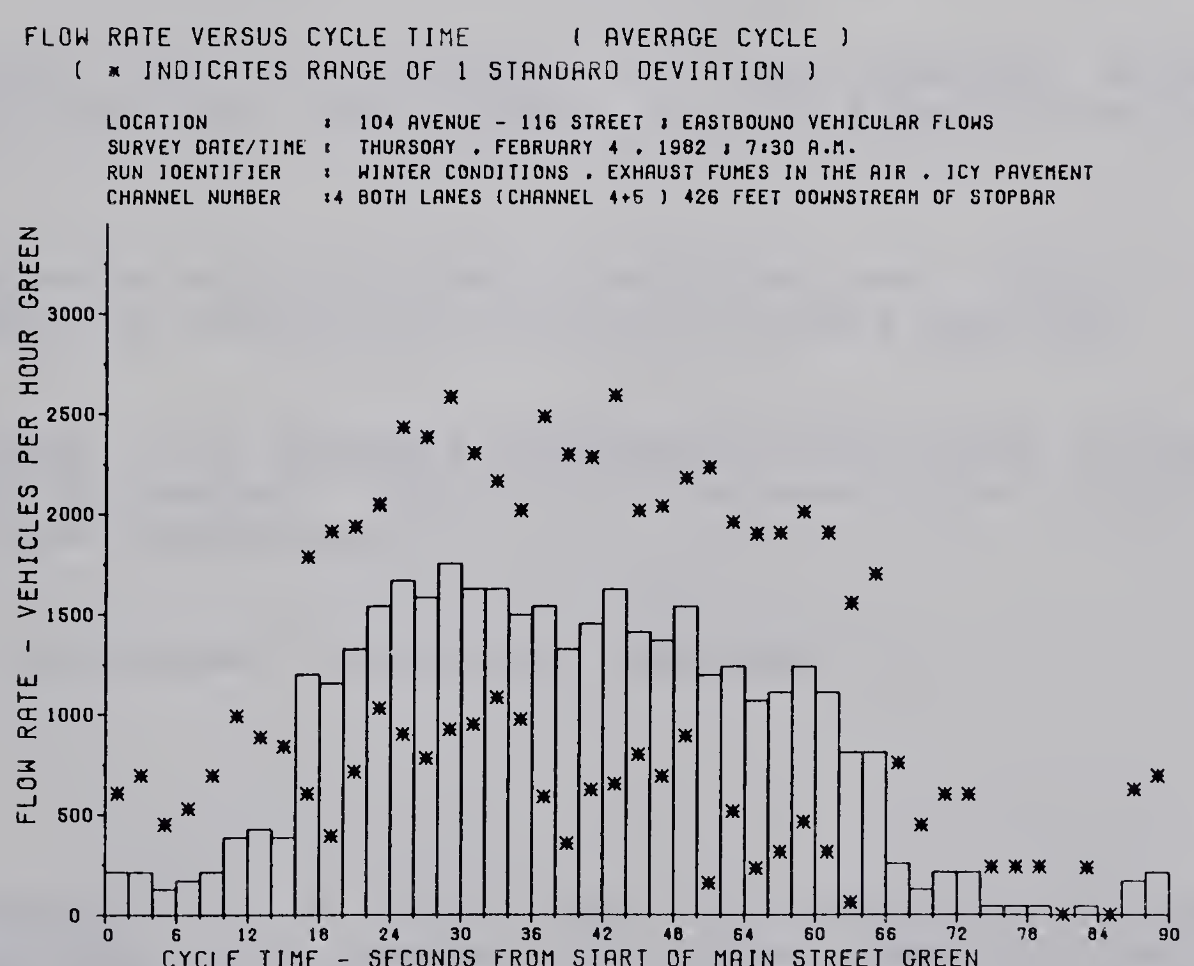


FIGURE 3.9

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION FOR 104 AVENUE - 116 STREET (SEVERE WINTER CONDITIONS)

1. Organization of data base for future reference : LOCATION: 104 Avenue -116 Street; Eastbound Vehicular flows. SURVEY DATE/TIME: Wednesday, March 10,1981; 7:30 A.M., RUN IDENTIFIER: Winter Month, Visual Inspection Of The Roadway Surface Would Suggest Typical Summer Conditions (Dry Pavement). CHANNEL NUMBERS: Locations #1 (Channel 0 + 1 Combined, 130 Metres (426 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 224 Metres (734 Feet) Downstream Of The Signal Stopbar).

2. Figure 3.8 illustrates a scaled schematic drawing of the 104 Avenue - 116 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a distance of 94 metres) is 8.24 seconds, 37.40 feet per second or 25.50 miles per hour.

4. A total of 28 cycles of data were collected with an average volume per cycle of 44.57 vehicles for two lanes with an unbaised standard deviation of 8.09 vehicles.

5. Signal timing information at the upstream signalized intersection (104 Avenue - 116 Street): see Section 3.3.1.

6. Characteristics of the survey site: see Section 3.3.1. Degree of saturation during the survey was 88%.

7. Figure 3.10 presents graphically the survey information for 104 Avenue - 116 Street (Average Winter Conditions, Further Downstream).

3.2.3 104 Avenue - 116 Street (Week No. 1)

1. Organization of data base for future reference : LOCATION: 104 Avenue -116 Street; Eastbound Vehicular flows. SURVEY DATE/TIME: Thursday, April 16,1981; 7:30 A.M., RUN IDENTIFIER: Summer Conditions, Dry Pavement, CHANNEL NUMBERS: Location #1 (Channel 0 + 1 Combined, 24 Metres (80 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 130 Metres (426

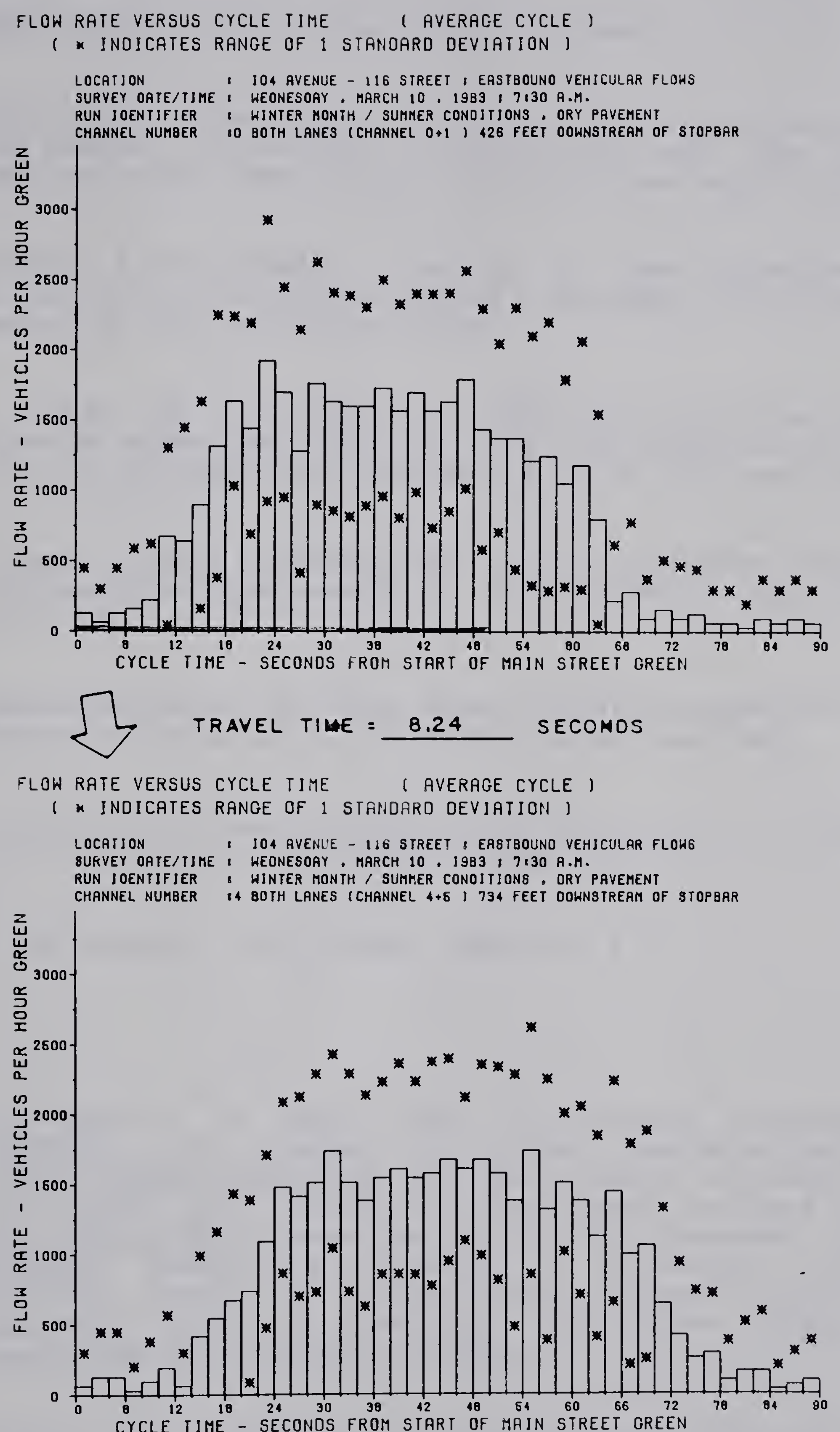


FIGURE 3.10

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION FOR 104 AVENUE - 116 STREET (AVERAGE WINTER CONDITIONS, FURTHER DOWNSTREAM)

Feet) Downstream Of The Signal Stopbar).

2. Figure 3.8 illustrates a scaled schematic drawing of the 104 Avenue - 116 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a distance of 106 metres) is 9.87 seconds, 35.05 feet per second or 23.9 miles per hour.

4. A total of 27 cycles of data were collected with an average volume per cycle of 48.81 vehicles for two lanes with an unbiased standard deviation of 4.59 vehicles.

5. Signal timing information at the upstream signalized intersection (104 Avenue - 116 Street): see Section 3.3.1.

6. Characteristics of the survey site: see Section 3.3.1. Degree of Saturation during the survey was 96%.

7. Figure 3.11 presents graphically the survey information for 104 Avenue - 116 Street (Week No. 1).

3.2.4 104 Avenue - 116 Street (Week No. 2)

1. Organization of data base for future reference : LOCATION: 104 Avenue -116 Street; Eastbound Vehicular flows. SURVEY DATE/TIME: Thursday, April 23,1981; 7:30 A.M.(Note: This survey was duplicated one week later), RUN IDENTIFIER: Summer Conditions, Dry Pavement, CHANNEL NUMBERS: Location #1 (Channel 0 + 1 Combined, 24 Metres (80 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 130 Metres (426 Feet) Downstream Of The Signal Stopbar).

2. Figure 3.8 illustrates a scaled schematic drawing of the 104 Avenue - 116 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a

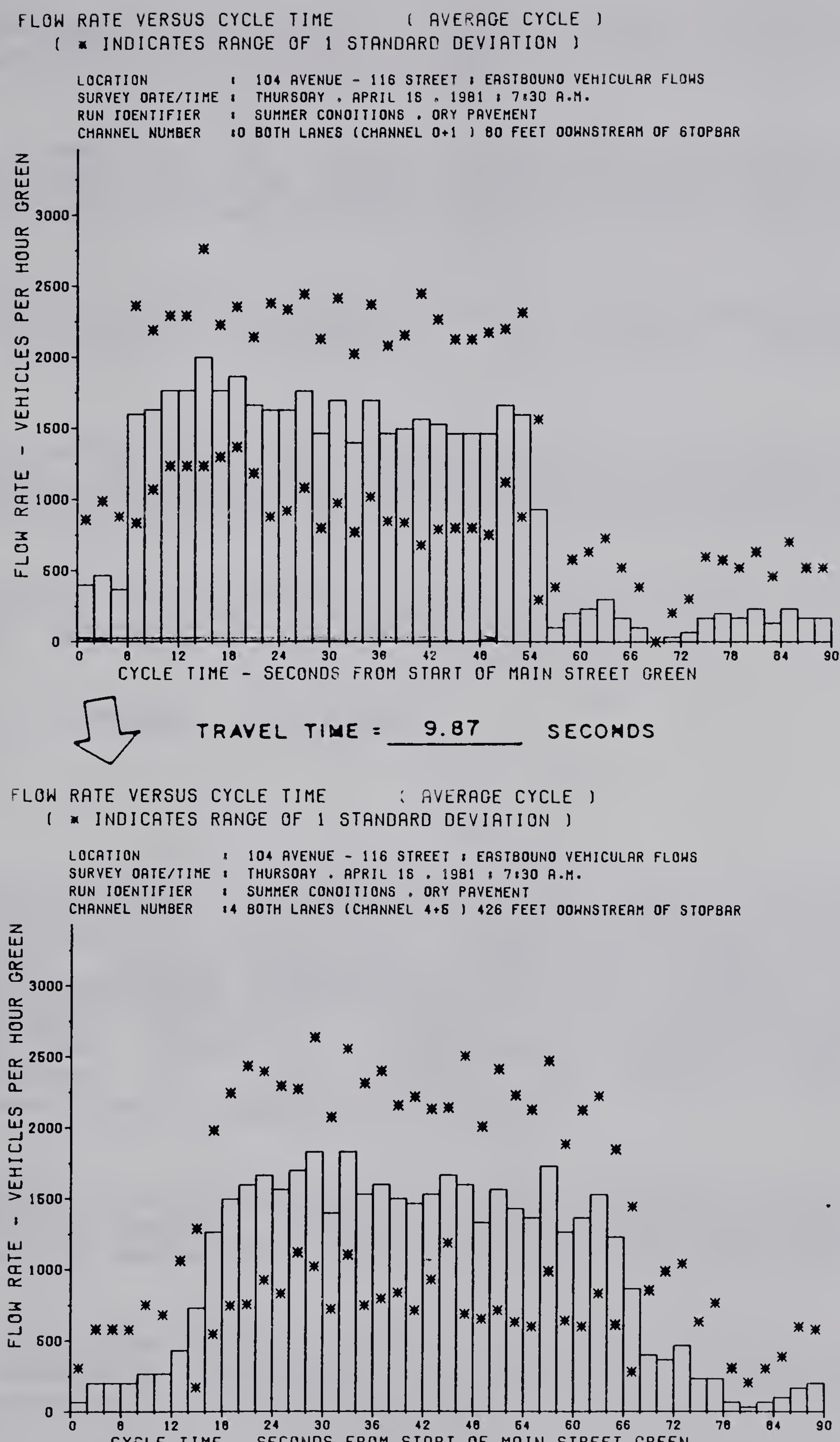


FIGURE 3.11

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION FOR 104 AVENUE - 116 STREET (WEEK NO. 1)

distance of 106 metres) is 9.87 seconds, 35.05 feet per second or 23.9 miles per hour.

4. A total of 27 cycles of data were collected with an average volume per cycle of 46.70 vehicles for two lanes with an unbiased standard deviation of 7.66 vehicles.

5. Signal timing information at the upstream signalized intersection (104 Avenue - 116 Street): see Section 3.3.1.

6. Characteristics of the survey site: see Section 3.3.1. Degree of Saturation during the survey was 92% (decrease of four percentage points from the previous survey).

7. Figure 3.12 presents graphically the survey information for 104 Avenue - 116 Street (Week No. 2).

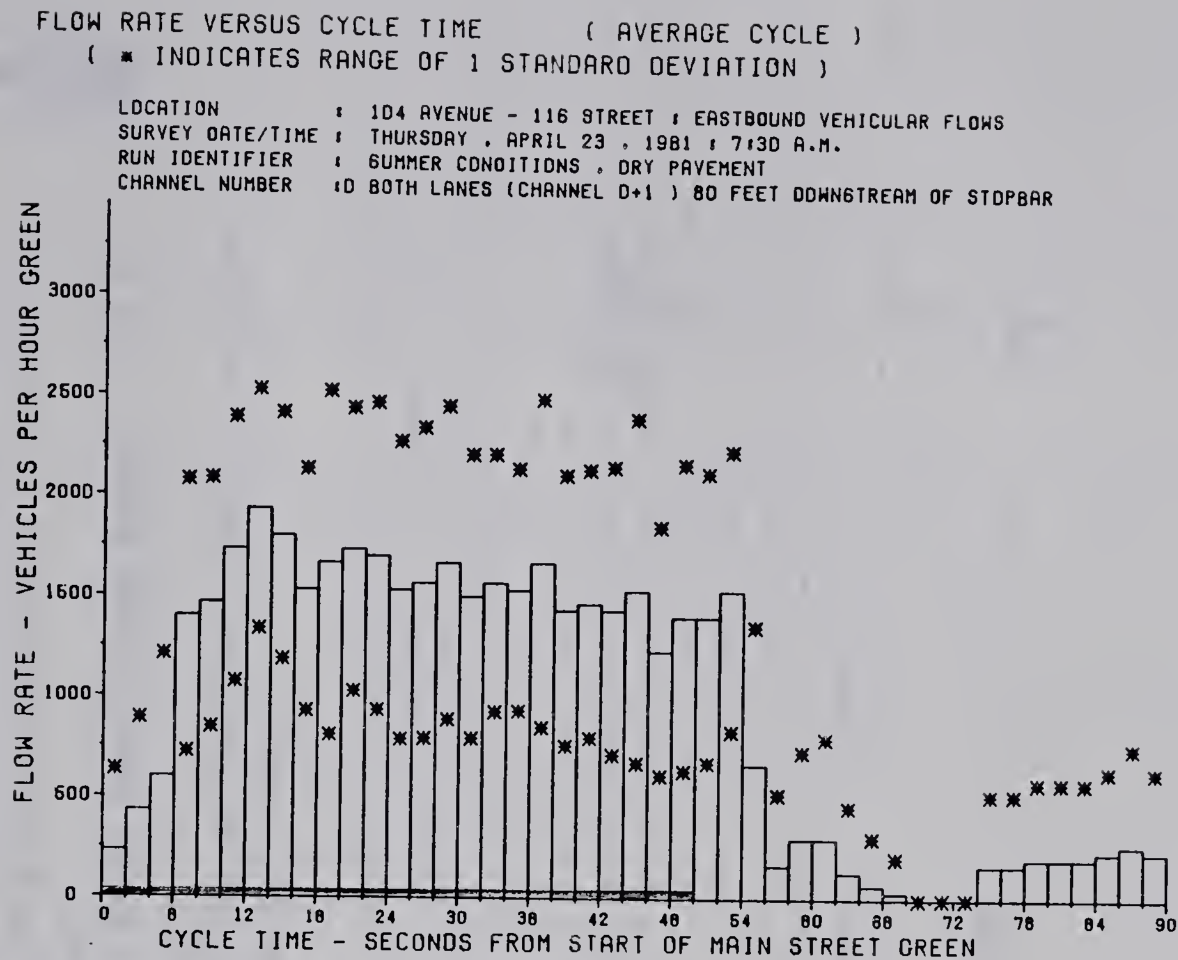
3.2.5 111 Avenue - 116 Street

1. Organization of data base for future reference : LOCATION: 111 Avenue -116 Street; Eastbound Vehicular flows. SURVEY DATE/TIME: Friday, May 6,1983; 7:30 A.M., RUN IDENTIFIER: Summer Conditions, Dry Pavement, CHANNEL NUMBERS: Location #1 (Channel 0 + 1 Combined, 24 Metres (80 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 130 Metres (426 Feet) Downstream Of The Signal Stopbar).

2. Figure 3.13 illustrates a scaled schematic drawing of the 111 Avenue - 116 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a distance of 106 metres) is 11.08 seconds, 31.24 feet per second or 21.30 miles per hour.

4. A total of 25 cycles of data were collected with an average volume per cycle of 15.44 vehicles for two lanes with an unbiased standard deviation of 4.06 vehicles.



TRAVEL TIME = 9.87 SECONDS

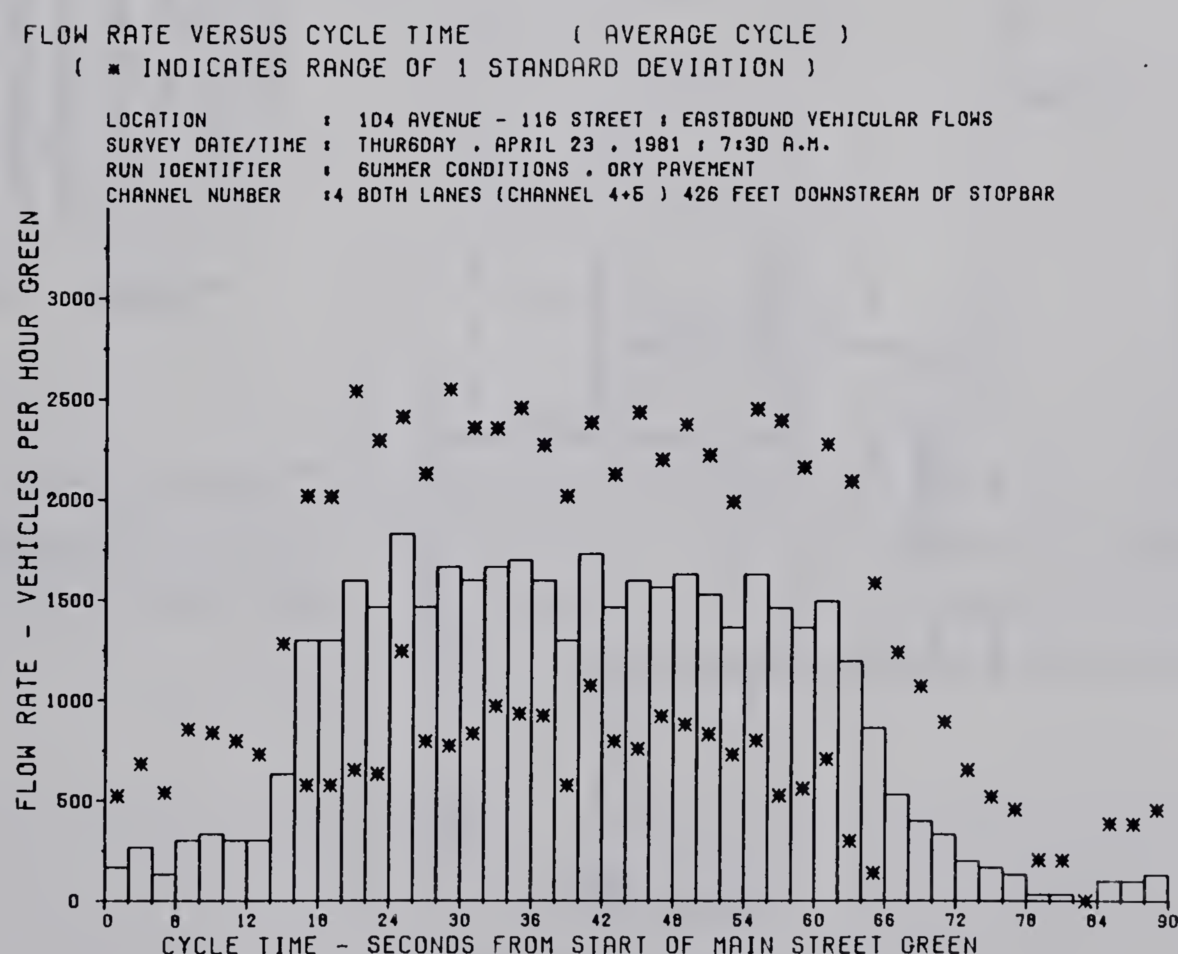


FIGURE 3.12

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION
FOR 104 AVENUE - 116 STREET (WEEK NO. 2)

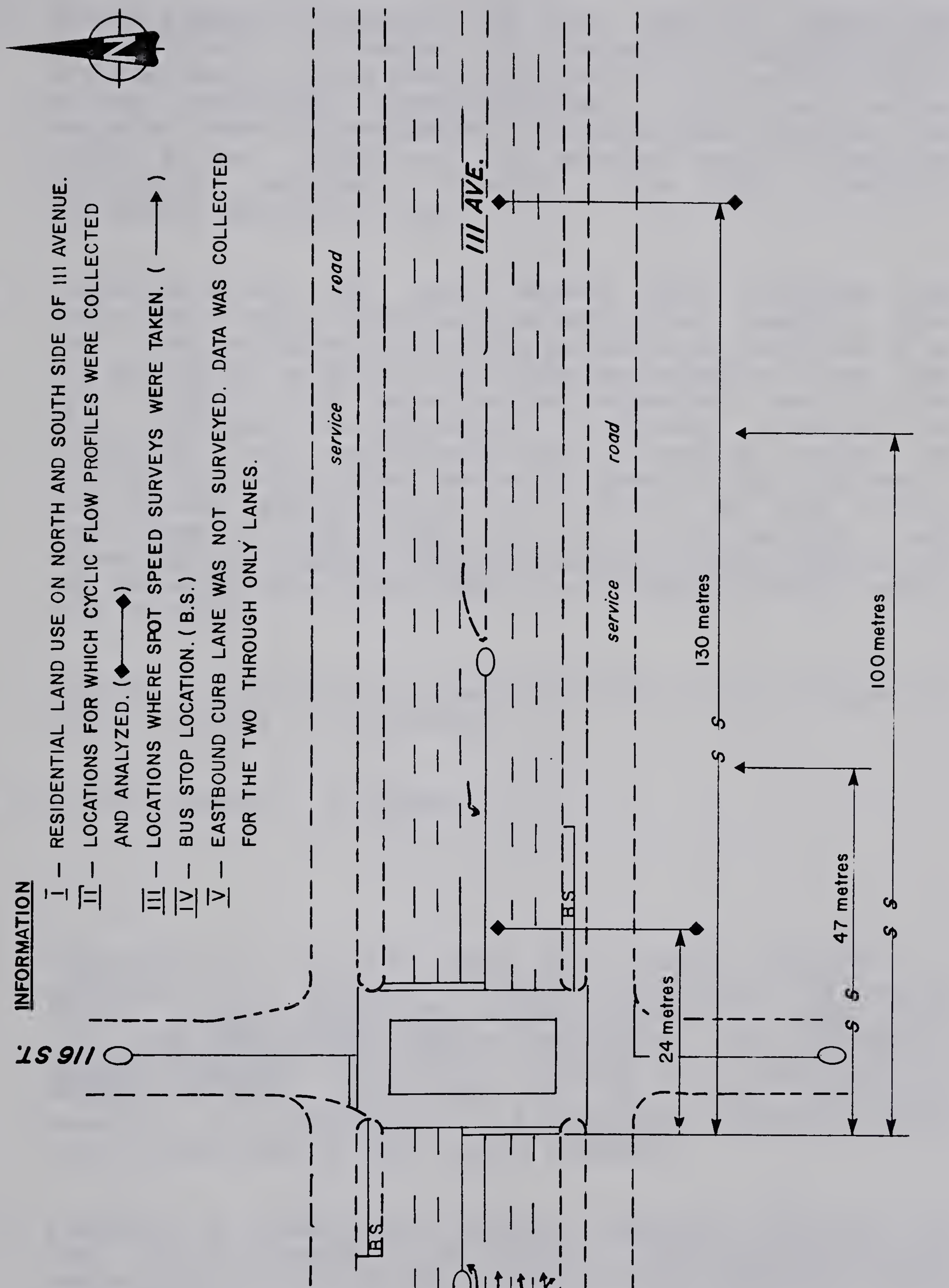


FIGURE 3.13

A SCALED SCHEMATIC DRAWING OF THE 111 AVENUE - 116 STREET INTERSECTION AND THE ROADWAY SECTION WHERE CYCLIC FLOW PROFILES WHERE COLLECTED

5. Signal timing information at the upstream signalized intersection (111 Avenue - 116 Street): cycle length=70 seconds, main street green (111 Avenue)=37 seconds, main street intergreen (3 seconds amber, 1 second all-red)=4 seconds, cross street green (116 street)=24 seconds, and cross street intergreen (3 seconds amber, 2 seconds all-red)=5 seconds. Interval size for analysis purposes=2 seconds=1 step.

6. Characteristics of the survey site includes the following : major arterial roadway system leading into CBD, medium degree of traffic pressure as measured by a 44% degree of saturation, no parking/stopping along the curb (note: curb lane was not surveyed), very little side friction (no driveway), passing freedom is generally not restricted due to medium degree of saturation (significant amount of passing was observed), bus volume approximately four buses per hour with a dwell time in the range of 10 - 15 seconds, and unrestricted traffic platoon movement (the location of the nearest downstream signal was approximately seven city blocks).

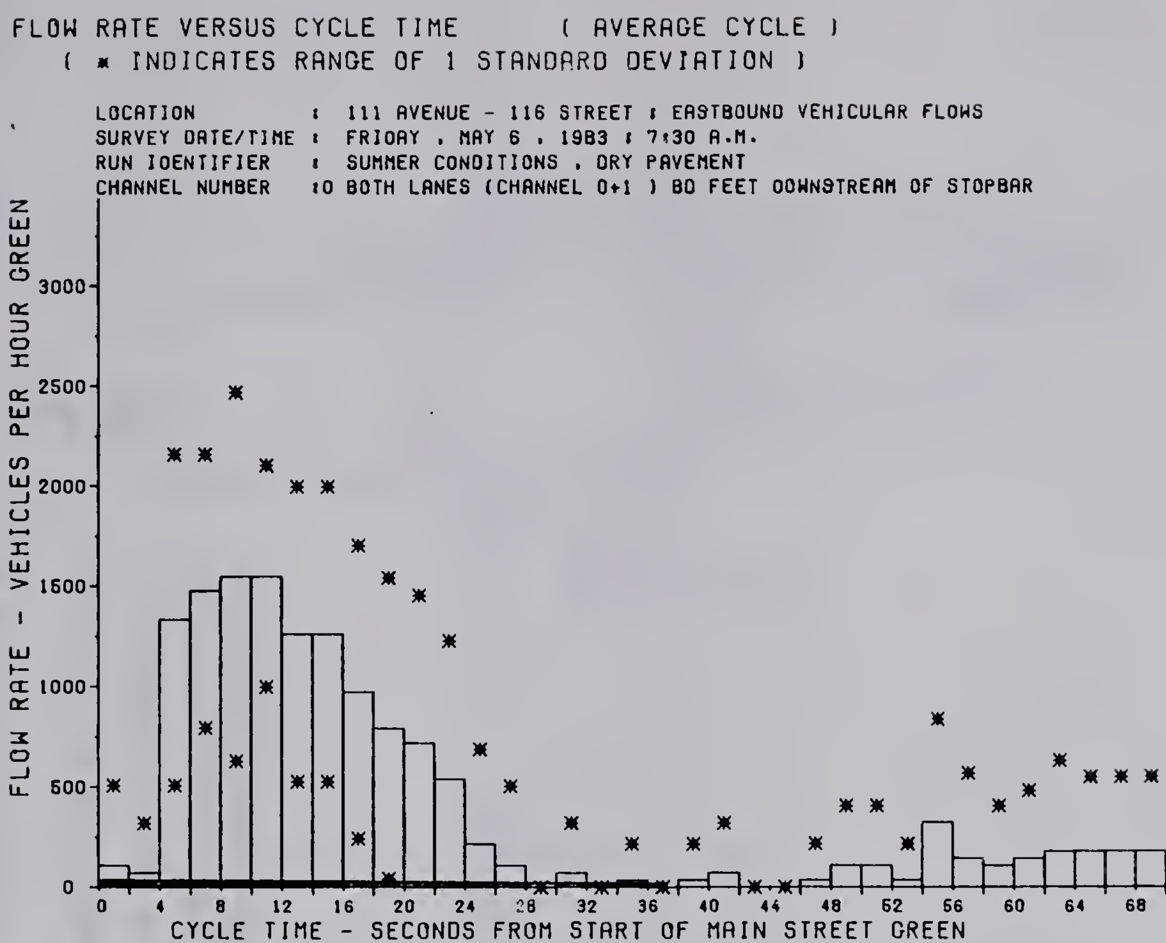
7. Figure 3.14 presents graphically the survey information for 111 Avenue - 116 Street.

3.2.6 102 A Avenue - 97 Street

1. Organization of data base for future reference : LOCATION: 102 A Avenue - 97 Street; Eastbound Vehicular flows. SURVEY DATE/TIME: Friday, October 14,1983; 7:25 A.M., RUN IDENTIFIER: Summer Conditions, Dry Pavement, CHANNEL NUMBERS: Location #1 (Channel 0 + 1 Combined, 21 Metres (70 Feet) Downstream Of The Signal Stopbar). Location #3 (Channel 4 + 5 Combined, 105 Metres (345 Feet) Downstream Of The Signal Stopbar).

2. Figure 3.15 illustrates a scaled schematic drawing of the 102 A Avenue - 97 Street intersection and the roadway section where these cyclic flow profiles were collected.

3. Travel time between Location #1 and Location #3 (a distance of 84 metres) is 12.76 seconds, 21.56 feet per second or 14.70 miles per hour (function of the



TRAVEL TIME = 11.08 SECONDS

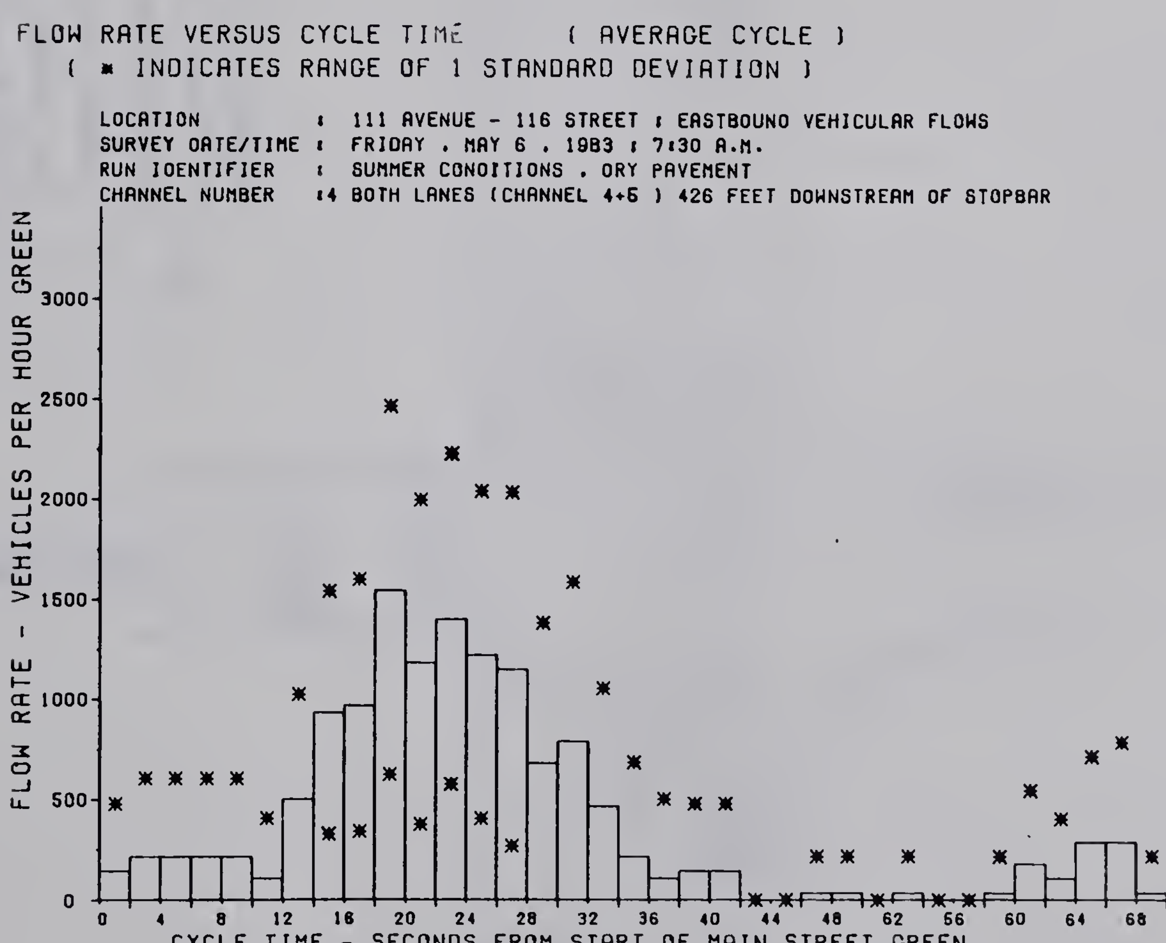


FIGURE 3.14

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION FOR 111 AVENUE - 116 STREET

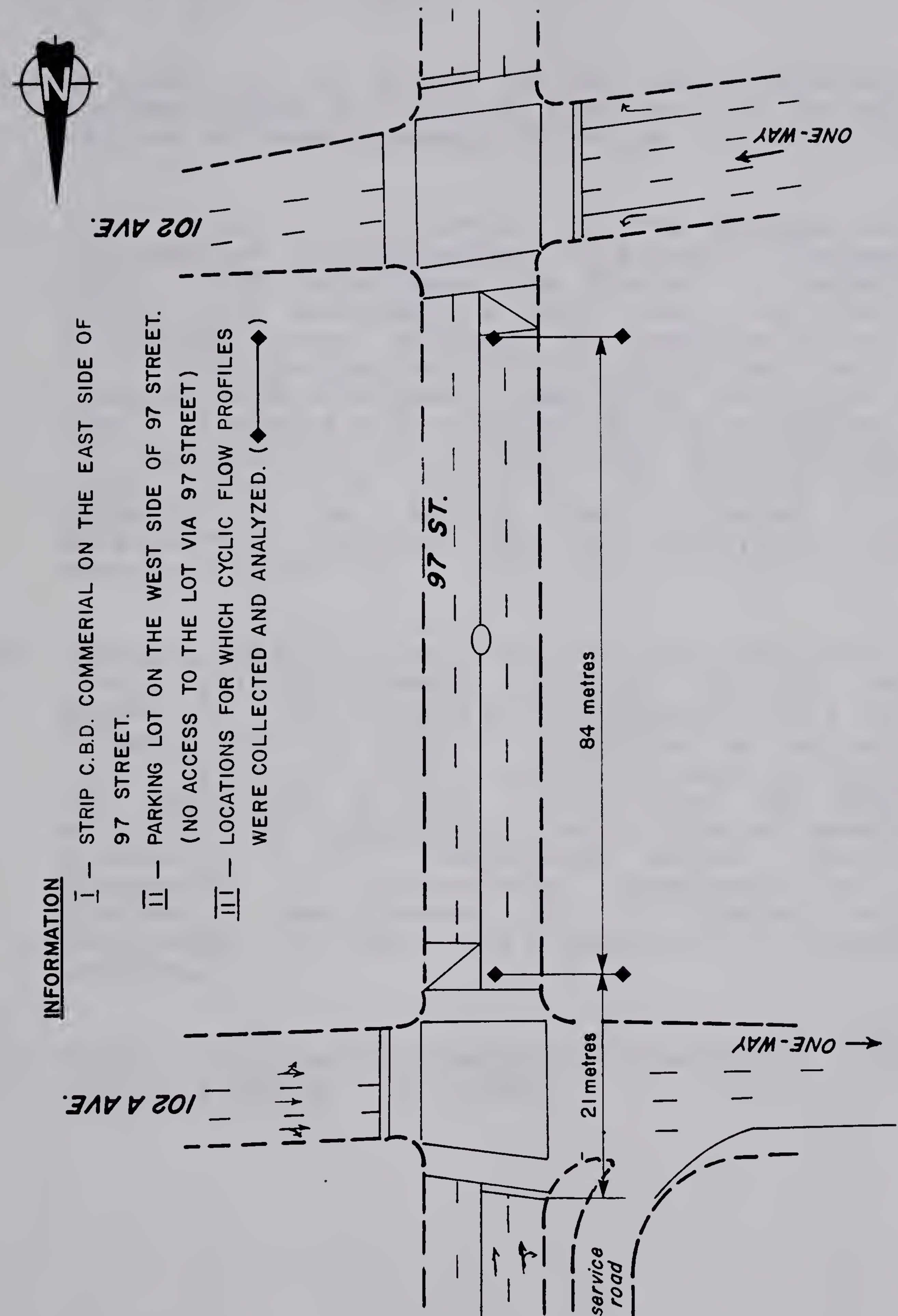


FIGURE 3.15

A SCALED SCHEMATIC DRAWING OF THE 102 A AVENUE - 97 STREET INTERSECTION AND THE ROADWAY SECTION WHERE CYCLIC FLOW PROFILES WERE COLLECTED

space-time diagram for southbound flows between 102 A Avenue and 102 Avenue).

4. A total of 23 cycles of data were collected with an average volume per cycle of 6.83 vehicles for two lanes with an unbiased standard deviation of 3.07 vehicles.

5. Signal timing information at the upstream signalized intersection (102 A Avenue - 97 Street) is presented in Figure 3.16 as a space-time diagram. In reviewing this figure it is important to note that, by design, the southbound green waveband is coordinated for a speed 23.7 kilometres per hour (14.7 miles per hour). Spot speed surveys undertaken immediately downstream of 102 A Avenue, indicate that vehicles desire to achieve a speed of approximately 35 kilometres per hour. Because of the signal design the phenonenon of platoon compression appears to be taking place (reverse of platoon dispersion). Interval size for analysis purposes=2 seconds=1 step.

6. Characteristics of the survey site include the following : major arterial roadway system in the CBD, very low degree of traffic pressure as measured by a 29% degree of saturation, no parking/stopping along the curb, very little side friction, passing freedom to manoeuvre is not restricted at all due to the low degree of saturation, and <u>restricted</u> traffic platoon movement (the proximity of the downstream signal affects the dispersion or alternately compression of traffic platoons). (Non-platoon to a platoon situation, compression of flow into a platoon or a re-grouping of vehicles.)

7. Figure 3.17 presents graphically the survey information for 102 A Avenue - 97 Street.

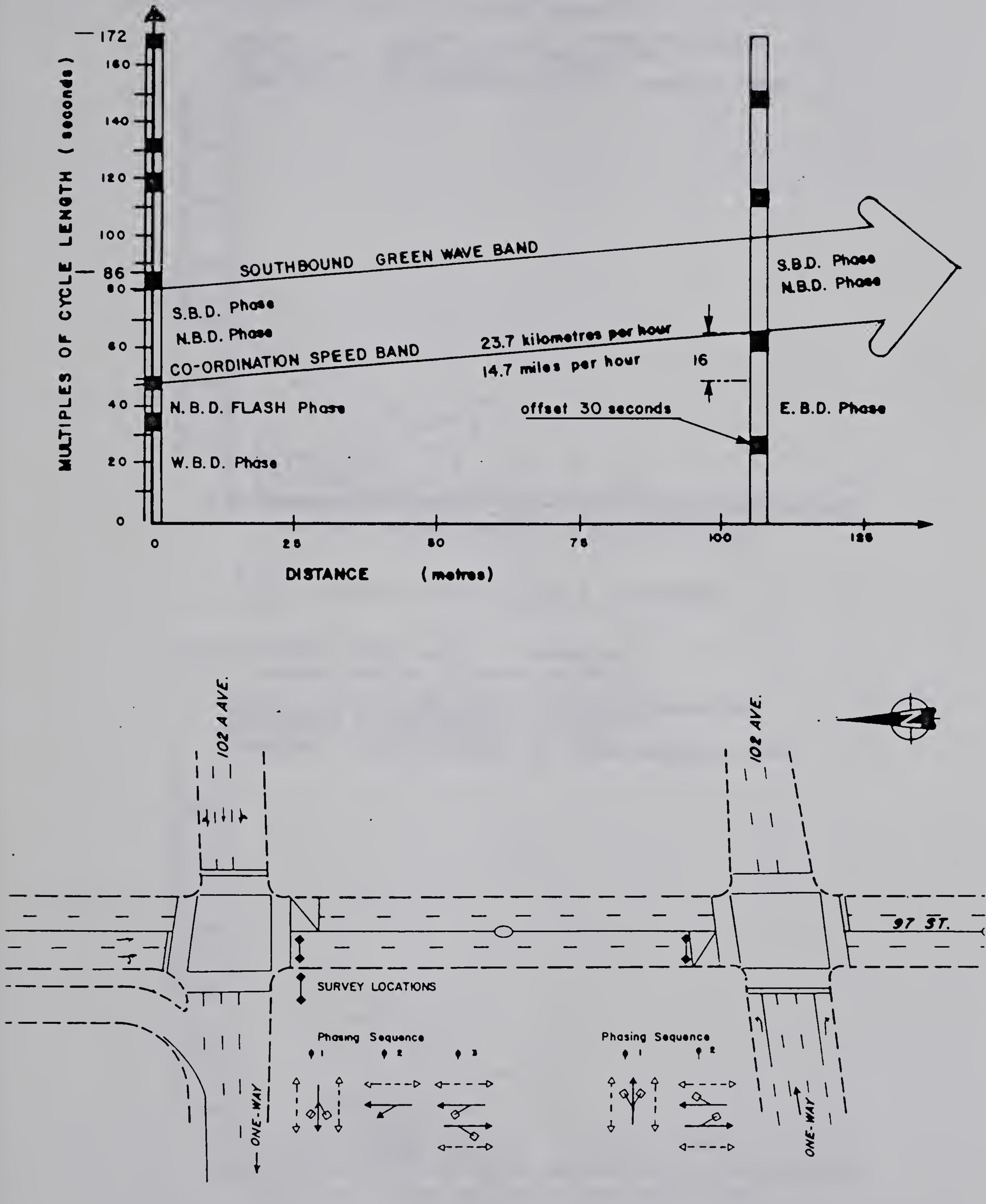


FIGURE 3.16

SPACE-TIME DIAGRAM ALONG 97 STREET (SOUTHBOUND) BETWEEN 102 A AVENUE AND 102 AVENUE

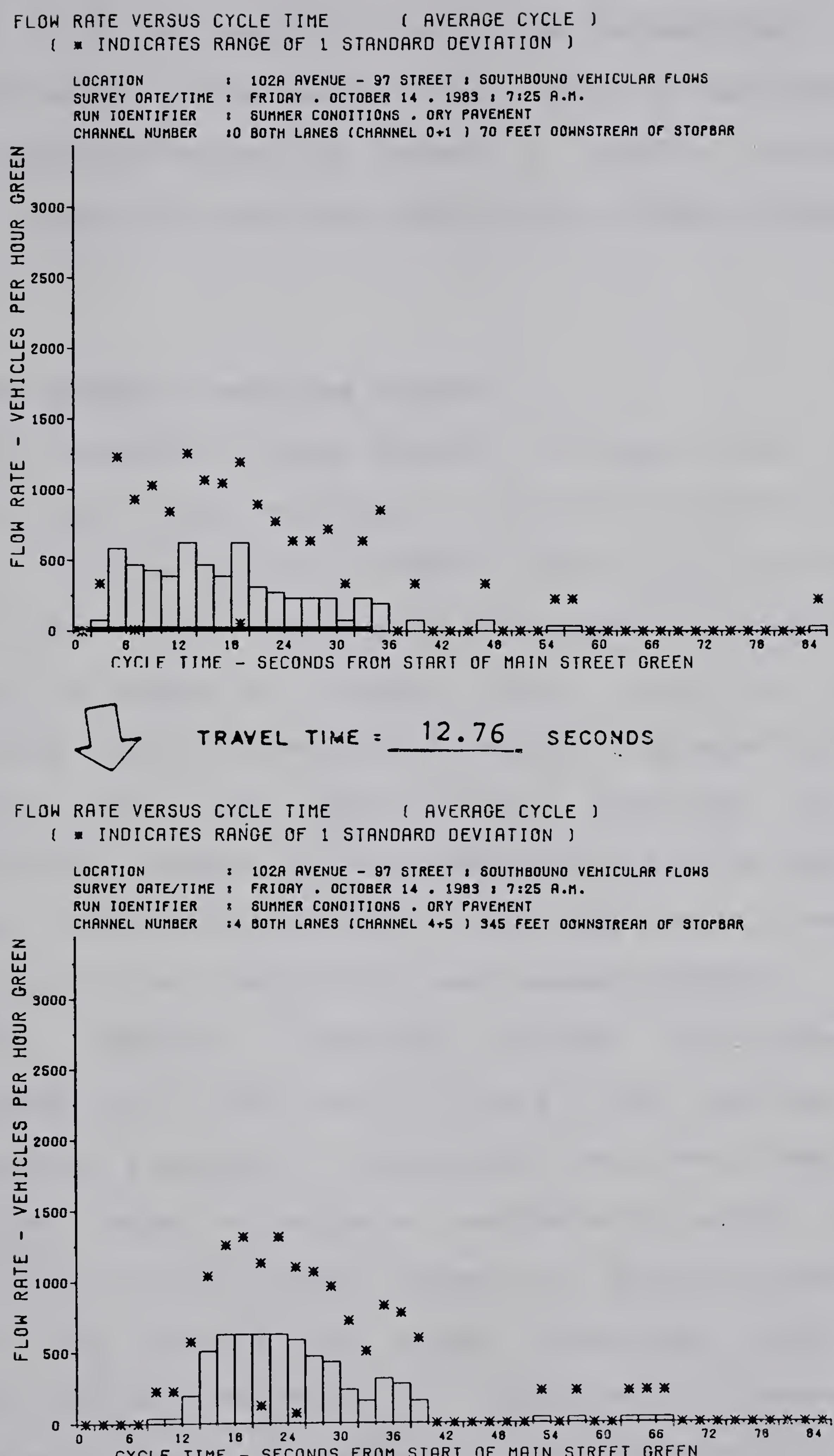


FIGURE 3.17

GRAPHICAL SUMMARY OF THE COLLECTED SURVEY INFORMATION FOR 102 A AVENUE - 97 STREET

4. THE ANALYTICAL TOOLS FOR CALIBRATION

To assist in the manipulation, handling, and management of the data collected with respect to model(s) calibration three computing programs were written. They are described below:

4.1 The GORDONV1 Computing Program

The computing program GORDONV1 is based on the program ROBERT, that was developed at the Toronto Traffic Control Centre ([29]). The version of ROBERT which was obtained was found, upon vigours testing to have several software errors (βT did not assume an integer value, prediction of the downstream cyclic flow profile was not displaced by βT time intervals and the delay/offset algorithm was not functioning). Based on the skeleton program of the ROBERT program, revisions where made to suit the specific needs of this research and the program was renamed GORDONV1.

The GORDONV1 computing program calculates the downstream cyclic flow profile from an input upstream cyclic flow profile according to Robertson's Recurrence Model for a specified range of alpha(α) and beta(β) values. The Root Mean Square Of Error (RMSE) between the predicted downstream cyclic flow profile and actual downstream cyclic flow profile over all intervals is calculated to serve as a preliminary means of measuring the goodness-of-fit. This provides a relative measure of the accuracy of the predicted cyclic flow profile compared to the actual cyclic flow

profile for a given street segment. It should be noted that this type of comparison would not be valid between different street segments.

The RMSE, which is one performance indicator to measure the accuracy of the Robertson model, takes the following form :

$$\text{RMSE} = \text{SQRT}\left[\sum_{i=1}^{n} [\text{Actual Flow}(i) - \text{Predicted Flow}(i)]^2 \right]$$

where :

RMSE = Root Mean Square Of Error

i = 1,2,...,n; where n=number of time intervals comprising the cycle length at the upstream signal

Actual Flow (i) = the observed flow in the i-th time interval at the downstream cyclic flow profile, and

Predicted Flow (i) = the predicted flow in the i-th time interval at the downstream cyclic flow profile.

For a given alpha value, the performance indicator RMSE, is calculated for a complete range of beta from a specified lower limit to a specified upper limit for a given increment size. Similarily, for a given beta value, the performance indicator RMSE, is calculated for a range of alpha from a specified lower limit to a specific upper limit for a given increment size. The analysis for each combination of alpha and beta is presented in a tabular and graphical form. The combination of alpha and beta which minimizes the RMSE performance indicator is therefore considered to be the 'best' calibrated empirical parameters.

A typical tabular summary of the GORDONV1 output is presented in Figure 4.1. The following information is summarized:

1. Robertson's beta value, βT (note the integer value in brackets), alpha value, and F factor.

2. Four header cards to organize the data base with respect to location, survey data/time, run identifier and channel number.

3. A table with five columns of data. Column 1 represents the number of intervals used in the analysis (interval size times number of intervals equal cycle length). Column 2 represents the initial upstream cyclic flow profile. This information is obtained from the arrival matrix (vehicles per interval size) for location #1. Column 3 represents the calculated downstream cyclic flow profile based on the empirical parameters at the top of the table and Robertson's Recurrence Model. Column 4 represents the actual/observed downstream cyclic flow profile. This information is obtained from the arrival matrix (vehicles per interval size) for location #3. Column 5 indicates for each interval the error between the actual downstream and predicted downstream flows.

In reviewing Figure 4.1, several items are worth noting as they relate to the data collection technique, and more importantly, to Robertson's Recurrence Model.

1. Generally speaking, the average cyclic volume for the initial cyclic flow profile should be equal to the actual downstream cyclic flow profile if no overcounting or undercounting takes place. Using the information presented in Figure 4.1 we can tabulate and find out that the average flow of the initial cyclic flow profile is 40.97 while the average flow of the actual downstream cyclic flow is 41.58. This discrepancy is due solely to an error in the data collection technique (surveyor error).

2. Consistently, the average cyclic volume for the predicted downstream cyclic flow profile will be less than the average cyclic volume for the initial cyclic

```
ROBERTSON'S EQUATION AS IT EXISTS IN THE TRANSYT MODEL
        T = 0.80 * AVG. JOURNEY TIME =    5.617 ( 6)
        F = 1/(1+ 0.50*T)= 0.250

LOCATION          : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME  : THURSDAY , FEBRUARY 4 , 1982 ; 7:30 A.M.
RUN IDENTIFIER    : WINTER CONDITIONS , EXHAUST FUMES IN THE AIR , ICY PAVEMENT
CHANNEL NUMBER    :  (0 +1) / (4 + 5) : 80 / 426 FEET DOWNSTREAM OF STOPBAR
```

INTERVAL NUMBER	INITIAL PLATOON	PREDICTED PLATOON	ACTUAL PLATOON	ERROR
1	0.38	0.12	0.24	0.12
2	0.48	0.14	0.24	0.10
3	0.29	0.15	0.14	-0.01
4	1.19	0.14	0.19	0.05
5	1.71	0.14	0.24	0.10
6	1.90	0.15	0.43	0.28
7	2.00	0.09	0.48	0.39
8	2.05	0.19	0.43	0.24
9	1.71	0.22	1.33	1.11
10	2.10	0.46	1.29	0.83
11	1.86	0.77	1.48	0.71
12	1.86	1.05	1.71	0.66
13	1.62	1.29	1.86	0.57
14	1.62	1.48	1.76	0.28
15	1.76	1.54	1.95	0.41
16	1.67	1.68	1.81	0.13
17	1.33	1.72	1.81	0.09
18	1.43	1.76	1.67	-0.09
19	1.48	1.72	1.71	-0.01
20	1.52	1.70	1 48	-0.22
21	1.29	1.71	1.62	-0.09
22	1.29	1.70	1.81	0.11
23	1.33	1.61	1.57	-0.04
24	1.24	1.56	1.52	-0.04
25	1.38	1.54	1.71	0.17
26	0.90	1.54	1.33	-0.21
27	1.19	1.48	1.38	-0.10
28	0.81	1.43	1.19	-0.24
29	0.05	1.40	1.24	-0.16
30	0.0	1.36	1.38	0.02
31	0.14	1.37	1.24	-0.13
32	0.05	1.25	0.90	-0.35
33	0.10	1.24	0.90	-0.34
34	0.0	1.13	0.29	-0.84
35	0.05	0.86	0.14	-0.72
36	0.05	0.64	0.24	-0.40
37	0.0	0.52	0.24	-0.28
38	0.05	0.40	0.05	-0.35
39	0.14	0.33	0.05	-0.28
40	0.14	0.24	0.05	-0.19
41	0.19	0.20	0.0	-0.20
42	0.19	0.16	0.05	-0.11
43	0.10	0.12	0.0	-0.12
44	0.14	0.10	0.19	0.09
45	0.19	0.11	0.24	0.13

FIGURE 4.1

TYPICAL TABULAR SUMMARY OF THE GORDONV1 OUTPUT

flow profile. This error can be calculated using the error equation presented in Section 2.3 where (1-F)/F takes on the value (1-0.25)/0.25 = 3.00 and

$$q'_{(n+\beta T)} = q'_{(6)}$$

takes on the value 0.15. The product of these two values (3.0 x 0.15 = 0.45) subject to rounding represents the absolute error. For this specific example we conclude that Robertson's equation underestimated the downstream average cyclic flow profile by 0.45 vehicles or alternatively by a factor of 1.01.

3. The calculated random delay for the average cyclic volume for the actual downstream cyclic flow profile is 6.08 seconds per vehicle. The calculated random delay for the average cyclic volume for the predicted downstream cyclic flow profile is 4.51 seconds per vehicle, or alternately, an underestimation of the random delay by a factor of 1.35. (See Section 2.3 and Secton 4.3).

4. For this specific example, Robertson's Recurrence Model will underestimate the predicted downstream volume by a factor of 1.01 which will in turn result in an underestimate of the random delay by a factor of 1.35.

Figure 4.2 illustrates a typical graphical summary of the goodness-of-fit between the predicted/actual downstream cyclic flow profile based on the information presented in Figure 4.1. The following specific information is noted:

1. Four header cards to organize the data base with respect to location, survey date/time, run identifier and channel number.

2. Plot of the measured/actual downstream cyclic flow profile which is indicated by an asterisk (*).

3. Plot of the predicted downstream cyclic flow profile which is indicated by a letter (o). A visual inspection of these graphs indicate that during severe winter conditions Robertson's recommended parameter of alpha=0.5 and beta=0.8 will produce significant error.

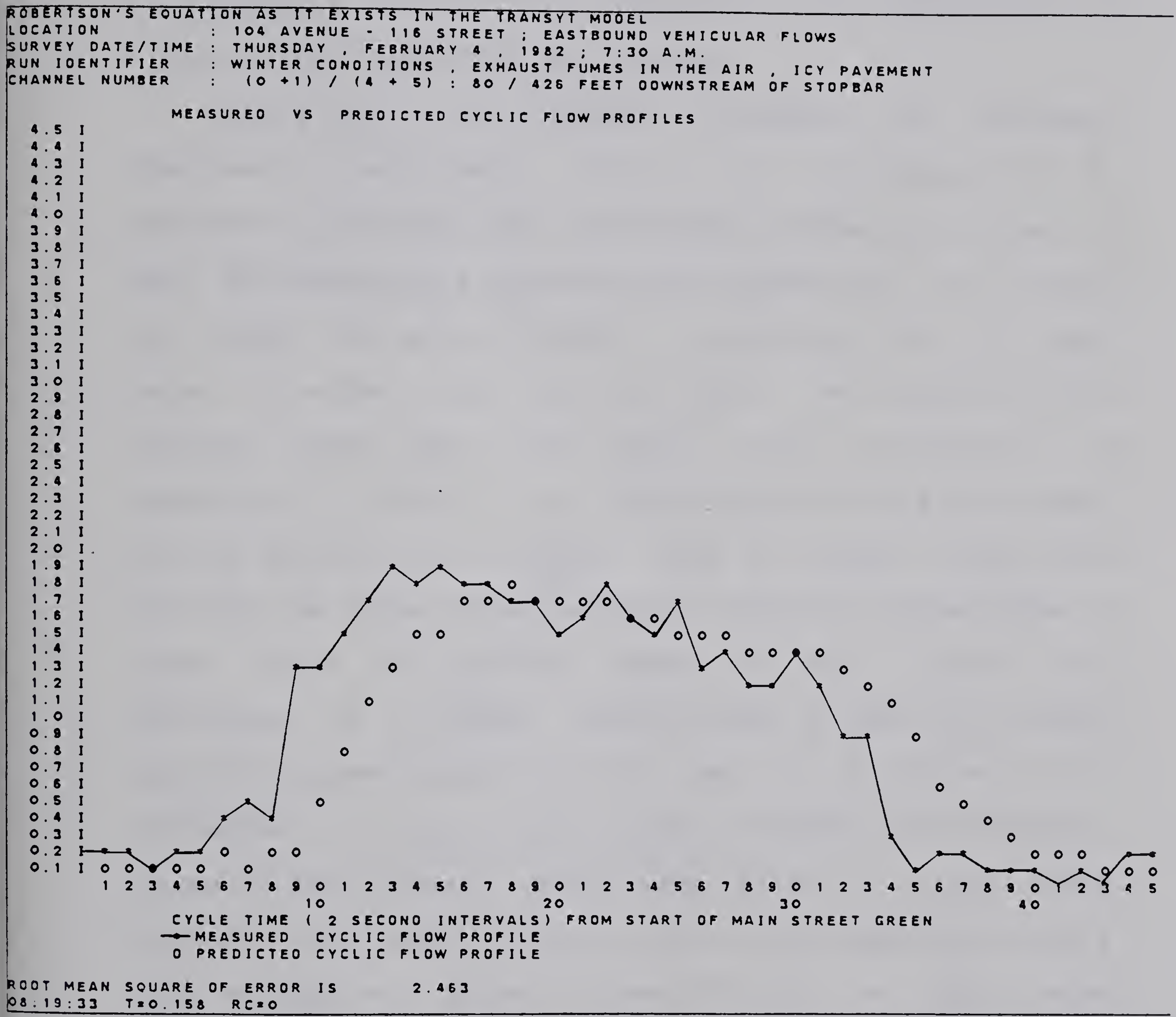


FIGURE 4.2

TYPICAL GRAPHICAL SUMMARY OF THE GORDONV1 OUTPUT

4. The X or horizontal axis is the cycle time in steps. The Y or vertical axis is flow per unit of time.

5. The RMSE performance indicator for the above two plots is summarized.

Figure 4.1 and 4.2 are repeated for all combination of alpha and beta specified by the user.

Before using the GORDONV1 program to calibrate Robertson's Recurrence Model, it is instructive to understand the functional relationship between the value of RMSE and Robertson's alpha and beta parameters. For a given beta value, the range of RMSE is calculated for a large range of alpha from 0.05 to 0.60. The results of this analysis, using data from severe winter conditions, is presented in Figure 4.3. Several observations are noted. Firstly there exists a unique value of alpha, which can minimize the global RMSE. Secondly, for this unique value of alpha, there are several values of beta which also contribute to a global minimal value of RMSE. One unique family of curves exists for each set of βT values which correspond to one integer value. Thirdly, for different ranges of beta values, there also exists a value which minimizes the RMSE; however, this value represents only a local minimum vs a global minimum. Forthly, an alpha value of 0.40 vs the standard value of 0.50 minimizes the value of RMSE.

For a given alpha value, the range of RMSE has also been calculated for a large range of beta from 0.30 to 0.80. The results of this analysis are presented in Figure 4.4.

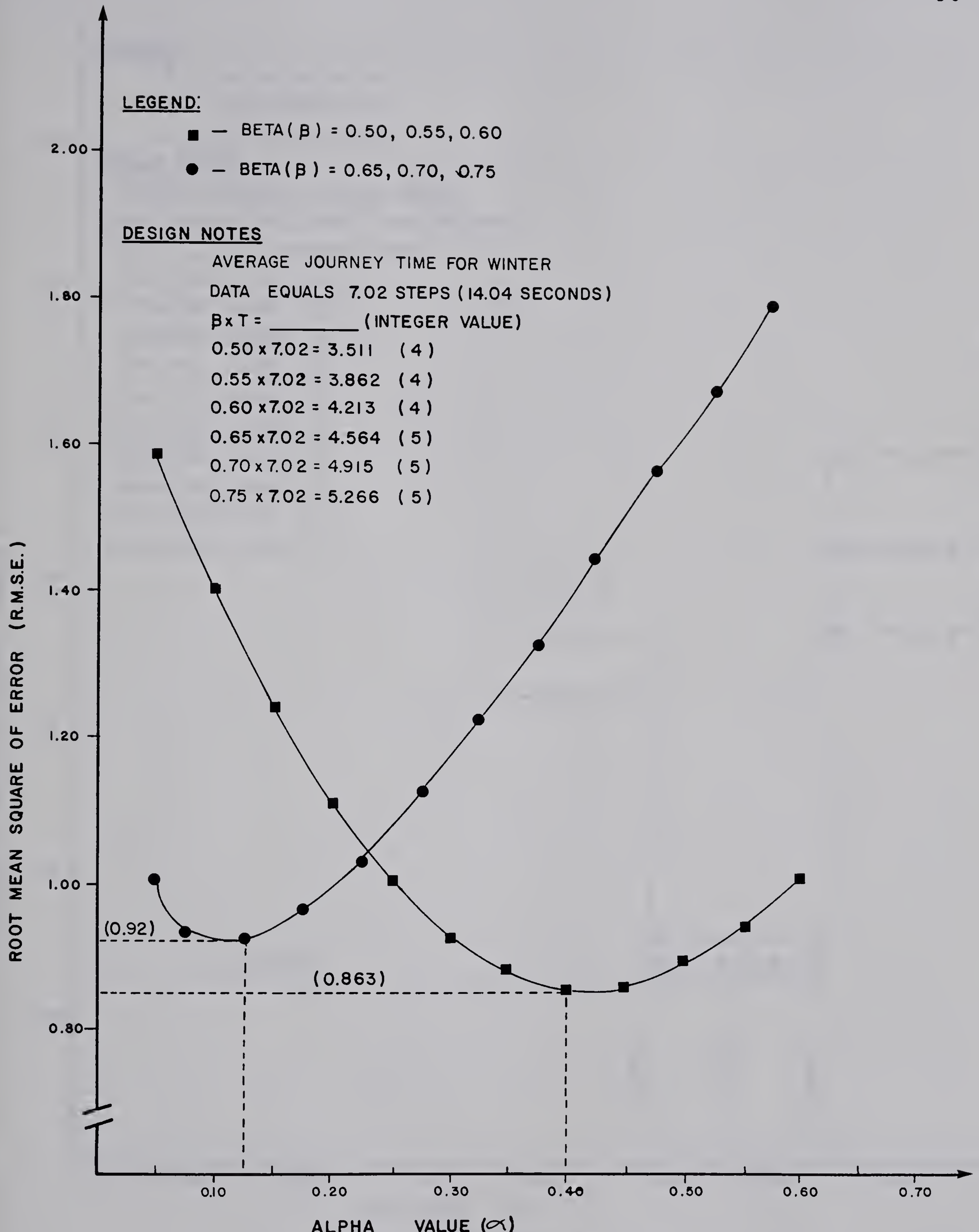


FIGURE 4.3

FUNCTIONAL RELATIONSHIP BETWEEN THE ALPHA VALUE AND ROOT MEAN SQUARE OF ERROR FOR VARIOUS BETA VALUES

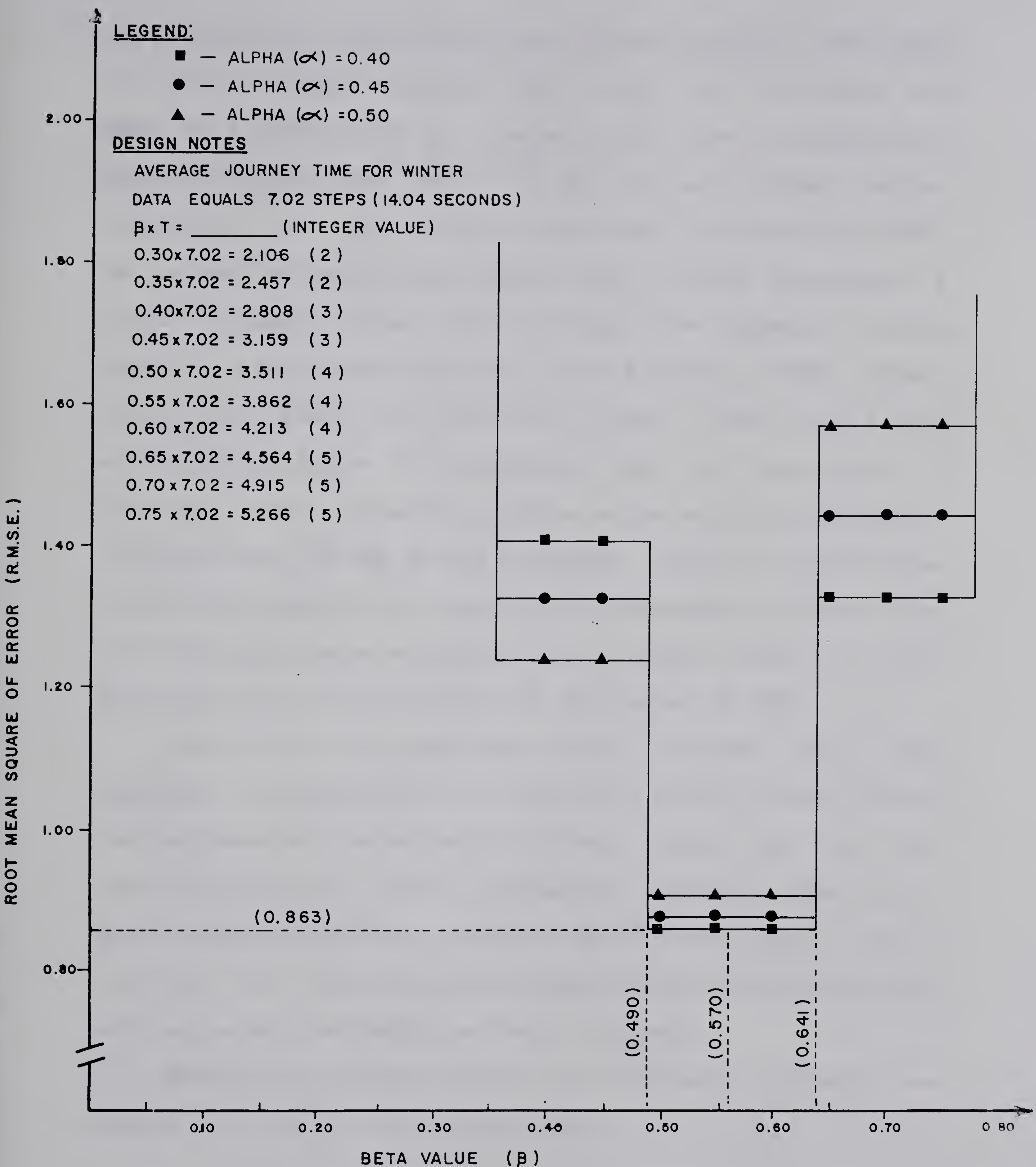


FIGURE 4.4

FUNCTIONAL RELATIONSHIP BETWEEN THE BETA VALUE AND ROOT MEAN SQUARE OF ERROR FOR VARIOUS ALPHA VALUES

The following observations are noted. Firstly, there does not exist a unique value of beta which can minimize the RMSE. This observation is related to the form of Robertson's equation in that the value of βT is an integer value. Therefore, for the range of beta that minimizes the RMSE, the product of beta times travel time in steps represents a unique integer value. This analysis also suggests for the range of beta values which will give a minimal RMSE error, there will also exist over that range of beta values only one smoothing factor F. We conclude that the selection of the best beta value is the beta value which minimizes the rounding error of the product of beta, times the travel time in seconds (equals an integer value). Secondly, a beta value of 0.570 (this value minimizes the rounding error) vs the standard value of 0.80 minimizes the value of RMSE.

The effect of alpha and beta on RMSE is a very important consideration in the calibration process, due to the complementary relationship between alpha, beta and the smoothing factor. This information highlights some of the deficiencies in previous research efforts(see Table 1.1). In specific, it is noted that those studies which calibrated only alpha may not have the 'best' values.

Based on the above analysis the following procedure was adopted for the calibration process:

1. Search for the optimal combination of alpha and beta which will minimize the RMSE.

2. Select the alpha value which minimizes the RMSE.

3. For the minimal RMSE select a beta value which has the least round-off error. (beta times travel time in steps equals an integer value).

4. The product of the selected alpha and beta value is referred to a K.

4.2 The GORDONV2 Computing Program

The GORDONV2 computing program is similar to the GORDONV1 computing program with one exception. The exception is that Robertson's Recurrence Model is modified by the introduction of an initialization equation as shown in Section 2.3. The program outputs and analysis process are identical to GORDONV1. In all cases it should be noted that the best combination of alpha and beta with the V1 program are identical to the best combination with the V2 program. There is one difference in that V2 will consistently result in a lower RMSE when compared to V1 since V2 predicted downstream volumes are identical to actual downstream volumes.

To distinguish the difference between V1 and V2 the following comments are printed on the top of each tabular and graphical output : <u>GORDONV1</u> - ROBERTSON'S EQUATION AS IT EXISTS IN THE TRANSYT PROGRAM, <u>GORDONV2</u> - MODIFIED ROBERTSON'S EQUATION.

4.3 The (DSPI) DELAY/STOP/PERFORMANCE INDEX-OFFSET Computing Program

The DSPI computing program calculates, for a given cyclic flow profile (basic input) and an artifical fixed time traffic signal, several signal performance characteristics. The computation of delays, stops, and performance index from a cyclic flow profile is based on relationships developed by Gartner([30]) and was validated on Bloor Street in Toronto. In specific, for each possible signal offset, the following performance indicators are calculated:

1. Average delay per vehicle (uniform plus random delay component).

2. Average stops per vehicle.

3. Performance index (weighted combination of stops and delay). In specific, the link performance index was established by summing the uniform delay component due to the signal design based on an average pattern plus the random delay component plus a stop penalty factor multiplied by the number of vehicle stops. For this research a stop penalty factor of 4 was used to allow for economic loss due to vehicle stoppings ([36]).

A typical computing output for the first page is illustrated in Figure 4.5. The following specific information is summarized :

1. Four header cards to organize the data base with respect to location, survey date/time, run identifier and channel number.

2. Echo of input data: cycle length, effective green (green plus 1 second), interval size, release or saturation rate (maximum discharge across a signalized stopline

```
          D E L A Y   O F F S E T   A N A L Y S I S          DATE:JAN 11, 1984
          =========================================
--------------------------------------------------------------------------------

 LOCATION            : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS

 SURVEY DATE/TIME    : THURSDAY , FEBRUARY 4 , 1982 ; 7:30 A.M.

 RUN IDENTIFICATION  : WINTER CONDITIONS , EXHAUST FUMES IN THE AIR , ICY PAVEMENT

 CHANNEL NUMBER      : CHANNEL(4+5) 426 FEET DOWNSTREAM OF STOPBAR <OBSERVED DATA>
--------------------------------------------------------------------------------

        I N P U T   D A T A

CYCLE LENGTH     =   90 SECONDS

EFFECTIVE GREEN  =   50 SECONDS

INTERVAL SIZE    =    2 SECONDS

RELEASE RATE     =3240. EPCU/HOUR

                 = 1.80 EPCU/INTERVAL

K-FACTOR         =   4.

ARRIVAL DATA     = 0.24 0.24 0.14 0.19 0.24 0.43 0.48 0.43 1.33 1.29

                   1.48 1.71 1.86 1.76 1.95 1.81 1.81 1.67 1.71 1.48

                   1.62 1.81 1.57 1.52 1.71 1.33 1.38 1.19 1.24 1.38

                   1.24 0.90 0.90 0.29 0.14 0.24 0.24 0.05 0.05 0.05

                   0.0  0.05 0.0  0.19 0.24
--------------------------------------------------------------------------------

        O U T P U T   D A T A

CUMMULATIVE ARRIVALS  = 41.58

RANDOM DEALY          =   2.81 VEHICLE HOURS PER HOUR

                      =   6.08 SECONDS PER VEHICLE

SATURATION RATIO      =   0.924
```

FIGURE 4.5

TYPICAL TABULAR SUMMARY OF THE DSPI OUTPUT (PAGE 1/2)

under the given traffic environmental conditions), K factor to weight the relative importance of stops/delay, and arrival data (obtained from the arrival matrix in the GORDONV1 or GORDONV2 computing outputs).

3. Brief overview of some output data including cumulative average cyclic arrivals, the TRANSYT (V5 - V6) random delay component and the degree of saturation expressed as a saturation ratio.

A typical computing output for the second page is illustrated in Figure 4.6. Basic information extracted from this output is indicated by an asterisk(*) and includes the following :

1. The offset (indicated in steps vs seconds) for which delays, stops or the performance index are minimized.

2. The associated absolute values of these three signal performance indicators.

The DSPI computing program was then used to evaluate at the downstream survey location the signal performance characteristics for the following types of cyclic flow profiles.

1. Measured/observed data.
2. A Robertson profile which was not calibrated.
3. A calibrated Robertson profile.
4. A calibrated, modified Robertson profile.

The principle purpose of this program is to be able to quantify the impacts in signal performance as a result of an improvement in the model and/or local calibration. The errors in signal performance by not calibrating and/or improving the model can be highlighted.

SUMMARY OF ANALYSIS DATE: JAN 11, 1984

INTER VAL	UNIFORM DELAY(SEC)	TOTAL DELAY(SEC)	AV DELAY /VEHICLE	# OF STOPS PER CYCLE	AVERAGE STOPS /VEHICLE	PERFORMANCE INDEX
1	1002.15	1254.69	30.18	41.15	0.99	15.77
2	1063.61	1316.55	31.66	41.10	0.99	16.46
3	1125.37	1378.11	33.14	41.10	0.99	17.14
4	1195.53	1448.27	34.83	41.20	0.99	17.92
5	1261.59	1514.33	36.42	41.25	0.99	18.66
6	1323.45	1576.19	37.91	41.15	0.99	19.34
7	1368.77	1621.51	39.00	40.91	0.98	19.83
8	1409.31	1662.05	39.97	40.67	0.98	20.27
9	1453.66	1706.41	41.04	40.67	0.98	20.77
10	1421.14	1673.89	40.26	39.34	0.95	20.35
11	1391.46	1644.21	39.54	38.53	0.93	19.98
12	1345.71	1598.45	38.44	37.05	0.89	19.41
13	1280.91	1533.65	36.88	35.34	0.85	18.61
14	1204.35	1457.09	35.04	35.34	0.85	17.76
15	1135.43	1388.17	33.39	35.34	0.85	16.99
16	1052.67	1305.41	31.40	35.34	0.85	16.08
17	980.31	1233.05	29.66	35.34	0.85	15.27
18	907.99	1160.73	27.92	35.34	0.85	14.47
19	844.95	1097.69	26.40	35.34	0.85	13.77
20	778.89	1031.64	24.81	35.34	0.85	13.03
21	727.37	980.12	23.57	33.86	0.81	12.40
22	666.53	919.28	22.11	32.24	0.78	11.65
23	593.96	846.70	20.36	32.24	0.78	10.84
24	535.48	788.22	18.96	32.67	0.79	10.21
25	479.72	732.46	17.62	31.15	0.75	9.52
26	412.76	665.50	16.01	29.44	0.71	8.70
27	366.14	618.88	14.88	28.11	0.68	8.13
28	315.98	568.72	13.68	26.73*	0.64*	7.51
29	275.90	528.64	12.71	28.16	0.68	7.13
30	233.36	486.10	11.69	28.40	0.68	6.66
31	186.56	439.30	10.57	30.21	0.73	6.22
32	154.26	407.00	9.79	30.59	0.74	5.88*
33	151.20	403.94	9.71	35.63	0.86	6.07
34	150.46*	403.20*	9.70*	35.92	0.86	6.08
35	201.74	454.48	10.93	39.49	0.95	6.80
36	270.26	523.00	12.58	41.15	0.99	7.64
37	331.92	584.66	14.06	41.20	0.99	8.33
38	393.68	646.42	15.55	41.10	0.99	9.01
39	471.58	724.32	17.42	41.29	0.99	9.88
40	550.02	802.76	19.31	41.53	1.00	10.77
41	628.68	881.42	21.20	41.53	1.00	11.64
42	711.73	964.48	23.20	41.58	1.00	12.56
43	790.49	1043.24	25.09	41.53	1.00	13.44
44	873.55	1126.29	27.09	41.58	1.00	14.36
45	940.01	1192.75	28.69	41.39	1.00	15.09

RANDOM DELAY= 6.08 SECONDS PER VEHICLE SATURATION RATIO= 0.92

FIGURE 4.6

TYPICAL TABULAR SUMMARY OF THE DSPI OUTPUT (PAGE 2/2)

In summary, the DSPI program becomes the tool to quantify the implications of calibration or modifications. In contrast, a statistical test such as chi-squared, would only measure statisticaly how closely the predicted cyclic flow profile comes to the observed cyclic flow profile. This latter test, gives no consideration to the implications of the goodness-of-fit from the perspective of delays, stops and performance index.

5. DISPERSION OF TRAFFIC PLATOONS

5.1 The Process Of Calibrating Robertson's Recurrence Model

The process of calibrating Robertson's recurrence model is presented in the following sequencial steps:

1. For the measured/observed downstream cyclic flow profile, the DSPI computing program was used to generate delay/offset, stop/offset and performance index/offset curves. Based on these curves, the following information was summarized :

   - Offset at which the actual delay is a minimum, and the true value of the delay at its minimum value.

   - Offset at which the actual stops is a minimum, and the true value of the stop at its minimum value.

   - Offset at which the actual performance index is a minimum, and the true value of the performance index at its minimum value.

   In reviewing this information, it should be recognized that the offset value which minimizes delay does not imply that stops or the performance index are also minimized at the same offset value. This is due to the fact that, essentially, delays, stops and performance index are different measures of effectiveness for signal control. This observation can be confirmed by referring to Figure 4.6 which indicates that a different minimium signal offset value occurs for delay, stops and the performance index.

2. The GORDONV1 computing program was used to generate two specific analysis for each site. The first analysis involved the calculation of the downstream cyclic flow profile based on Robertson's recommended parameters of alpha=0.50 and beta=0.80 and the resulting RMSE statistic. The second analysis involved the use of the GORDONV1 program to search out the combination of alpha and beta which reuslts in a global minimum value of the RMSE performance indicator. This process generated a

standard Robertson profile and a calibrated downstream cyclic flow profile based on the Robertson's Recurrence Model. The DSPI computing program was used to generate a delay/stop/performance-index offset infomation. Based on these predicted cycle flow profiles and the actual cyclic flow profile, the following statistics were summarized in the tabular form :

- Offset at which the predicted delay from the predicted cyclic flow profile is a minimum, and the predicted value of the delay at its minimum. This predicted value of the delay is compared to the actual value of delay which is generated from the actual cyclic flow profile at the best offset given by the predicted cyclic flow profile.

- Offset at which the predicted stops from the predicted cyclic flow profile are a minimum, and the predicted value of the stops at its minimum. This predicted value of the stops is compared to the actual value of stops which is generated from the actual cyclic flow profile at the best offset given by the predicted cyclic flow profile.

- Offset at which the predicted performance index form the predicted cyclic flow profile is a minimum, and the predicted value of the performance index at its minimum. This predicted value of the performance index is compared to the actual value of the performance index which is generated from the actual cyclic flow profile at the best offset given by the predicted cyclic flow profile.

<u>For a perfect fit between the predicted downstream cyclic flow profile and the actual downstream cyclic flow profile, the differences between the various performance indicators at the best offset value or over all offset values would be zero</u>.

3. The previously described process was repeated with the GORDONV2 computing program with the exception that there was no need to search out the combination of alpha and beta which results in a global minimum value of the RMSE performance indicator.

5.2 Calibration Results

For each of the six survey sites, four cases are analyzed from the perspective of delays, stops and performance index. These cases are:

Case I : Robertson's Model (Robertson's recommended alpha value of 0.50 and beta value of 0.80 is assumed.)

Case II : Calibrated Robertson's Model (Calibration of both alpha and beta values).

Case III : Calibrated, Modified Robertson's Model (Introduction of an initialization equation and calibration of both alpha and beta values).

Case IV : Modified Robertson's Model (Robertson's recommended alpha value of 0.50 and beta value of 0.80 is assumed).

Tables 5.1 to 5.6 inclusive provide a comparison of platoon prediction errors with respect to delay, stops and performance index for all intersection surveyed. To explain in more detail the exact process and the results obtained, Table 5.1 will be explained in detail.

For the measured/observed downstream cyclic flow profile the minimum delay occurs at an offset of 68 seconds and the true value of this delay at its minimum value is 9.70 seconds per vehicle. Similarily, the minimum stops occurs at an offset of 56 seconds and the true value of the stops at its miminal value is 0.64 stops per vehicle. Also, the minimum performance index occurs at an offset of 64 seconds and the true value of the performance index at its

TABLE 5.1

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 104 AVENUE - 116 STREET (SEVERE WINTER CONDITIONS)

LOCATION : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : THURSDAY, FEBRUARY 4, 1982 ; 7:30 A.M.
RUN IDENTIFIER : SEVERE WINTER CONDITIONS, EXHAUST FUMES IN THE AIR, ICY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 128 METERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS		
	ALPHA = 0.40; BETA = 0.570	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delay			(68) 9.70
* Stops			(56) 0.64
* Performance Index			(64) 5.88
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.863	2.463	
* Predicted/Actual Delay	(66) 7.85/9.71	(72) 8.52/12.58	
* Predicted/Actual Stops	(60) 0.52/0.68	(64) 0.30/0.74	
* Predicted/Actual Performance Index	(66) 4.91/6.07	(68) 4.94/6.08	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.844	2.355	
* Predicted/Actual Delay	(66) 8.39/9.71	(72) 9.50/12.58	
* Predicted/Actual Stops	(60) 0.53/0.68	(62) 0.34/0.73	
* Predicted/Actual Performance Index	(66) 5.25/6.07	(70) 5.70/6.80	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

TABLE 5.2

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 104 AVENUE - 116 STREET
(AVERAGE WINTER CONDITIONS, FURTHER DOWNSTREAM)

LOCATION : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : WEDNESDAY, MARCH 10, 1983 ; 7:30 A.M.
RUN IDENTIFIER : WINTER MONTH / SUMMER CONDITIONS, DRY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 215 METERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS: ALPHA = 0.30; BETA = 0.725	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delay			(72) 6.35
* Stops			(62) 0.31
* Performance Index			(72) 4.14
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.850	0.944	
* Predicted/Actual Delay	(72) 5.70/6.35	(74) 6.07/6.50	
* Predicted/Actual Stops	(66) 0.22/0.32	(66) 0.26/0.32	
* Predicted/Actual Performance Index	(72) 3.63/4.14	(72) 3.81/4.14	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.847	0.938	
* Predicted/Actual Delay	(74) 5.78/6.35	(74) 6.22/6.50	
* Predicted/Actual Stops	(66) 0.22/0.32	(66) 0.26/0.32	
* Predicted/Actual Performance Index	(72) 3.76/4.14	(72) 3.96/4.14	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

TABLE 5.3

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 104 AVENUE - 116 STREET (WEEK NO. 1)

LOCATION : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : THURSDAY, APRIL 16, 1981 ; 7:30 A.M.
RUN IDENTIFIER : SUMMER CONDITIONS, DRY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 130 MATERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS		
	ALPHA = 0.45; BETA = 0.811	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delay			(68) 15.67
* Stops			(40) 0.78
* Performance Index			(68) 10.44
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	1.029	1.037	
* Predicted/Actual Delay	(66) 14.00/16.24	(68) 14.01/15.67	
* Predicted/Actual Stops	(48) 0.54/0.82	(48) 0.55/0.82	
* Predicted/Actual Performance Index	(66) 9.25/10.60	(68) 9.28/10.60	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	1.009	1.010	
* Predicted/Actual Delay	(66) 15.81/16.24	(68) 16.14/15.67	
* Predicted/Actual Stops	(46) 0.58/0.86	(44) 0.62/0.86	
* Predicted/Actual Performance Index	(66) 10.37/10.60	(66) 10.57/10.60	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

TABLE 5.4

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 104 AVENUE - 116 STREET (WEEK NO. 2)

LOCATION : 104 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : THURSDAY, APRIL 23, 1981 ; 7:30 A.M.
RUN IDENTIFIER : SUMMER CONDITIONS, DRY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 130 METERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS		
	ALPHA = 0.40; BETA = 0.811	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delay			(68) 8.73
* Stops			(58) 0.48
* Performance Index			(66) 5.93
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.829	0.893	
* Predicted/Actual Delay	(66) 8.50/8.84	(68) 8.61/8.73	
* Predicted/Actual Stops	(54) 0.39/0.51	(56) 0.39/0.51	
* Predicted/Actual Performance Index	(66) 5.72/5.93	(66) 5.73/5.93	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.788	0.832	
* Predicted/Actual Delay	(66) 9.20/8.84	(68) 9.50/8.73	
* Predicted/Actual Stops	(52) 0.43/0.51	(54) 0.44/0.51	
* Predicted/Actual Performance Index	(66) 6.22/5.93	(66) 6.38/5.93	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

TABLE 5.5

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 111 AVENUE - 116 STREET

LOCATION : 111 AVENUE - 116 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : FRIDAY, MAY 6, 1983 ; 7:30 A.M.
RUN IDENTIFIER : SUMMER CONDITIONS, DRY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 130 METERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS: ALPHA = 0.35; BETA = 0.722	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delay			44 (1.19)
* Stops			36 (0.10)
* Performance Index			44 (0.35)
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.659	0.775	
* Predicted/Actual Delay	(44) 1.44/1.19	(46) 1.64/1.40	
* Predicted/Actual Stops	(40) 0.10/0.10	(42) 0.11/0.10	
* Predicted/Actual Performance Index	(42) 0.40/0.37	(46) 0.45/0.42	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.640	0.734	
* Predicted/Actual Delay	(44) 1.43/1.19	(46) 1.63/1.40	
* Predicted/Actual Stops	(40) 0.10/0.10	(42) 0.10/0.10	
* Predicted/Actual Performance Index	(42) 0.40/0.37	(46) 0.46/0.42	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

TABLE 5.6

COMPARISON OF PLATOON PREDICTION ERRORS WITH RESPECT TO DELAY, STOPS AND PERFORMANCE INDEX FOR 102 A AVENUE - 97 STREET

LOCATION : 102 A AVENUE - 97 STREET ; EASTBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : FRIDAY, OCTOBER 14, 1983 ; 7:30 A.M.
RUN IDENTIFIER : SUMMER CONDITIONS, DRY PAVEMENT
CHANNEL NUMBER : 2 CHANNELS (4+5), 105 METERS DOWNSTREAM OF STOPBAR

MEASURES OF EFFECTIVENESS	ROBERTSON'S ALPHA AND BETA PARAMETERS		
	ALPHA = 0.15; BETA = 0.627	ALPHA = 0.50; BETA = 0.80	NOT APPLICABLE
Measured/Observed Platoon Data			
* Delays			(42) 1.45
* Stops			(42) 0.04
* Performance Index			(42) 0.13
Calibrated Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.621	1.034	
* Predicted/Actual Delay	(50) 2.87/2.81	(54) 4.57/5.40	
* Predicted/Actual Stops	(42) 0.13/0.28	(46) 0.20/0.16	
* Predicted/Actual Performance Index	(46) 0.32/0.18	(54) 0.48/0.68	
Calibrated Modified Robertson's Model (Robertson's Recommended Parameters Are Also Included)			
* Root Mean Square Of Error	0.620	1.028	
* Predicted/Actual Delay	(50) 2.89/2.81	(54) 4.60/5.40	
* Predicted/Actual Stops	(42) 0.13/0.28	(46) 0.20/0.16	
* Predicted/Actual Performance Index	(46) 0.32/0.18	(54) 0.49/0.68	

NOTE : Values In Brackets Represent Best Offset Values (In Seconds) For The Predicted Measures Of Effectiveness. The Corresponding Actual Measures Of Effectiveness Are At The Best Offset Value For The Predicted Measures Of Effectiveness.

minimum value is 5.88.

For the predicted downstream cyclic flow profile which is based on Robertson's Model (alpha=0.50, beta=0.80), the minimum delay occurs at an offset of 72 seconds and the predicted value of the delay at its minimum value is 8.52 seconds per vehicles. Using the observed downstream cyclic flow profile, the true delay at the 72 second offset value is 12.58 seconds per vehicle. Similarily, the minimum stops occurs at an offset of 64 seconds and the predicted value of the stops at its minimum value is 0.30 stops per vehicle . Using the observed downstream cyclic flow profile the true value of stops at the 64 second offset value is 0.74. Also, the minimal performance index occurs at an offset of 68 seconds and the predicted value of the performance index at its minimal value is 4.94. Using the observed downstream cyclic flow profile, the true performance index at the 68 second offset value is 6.08. In summary, for this particular site with the use of Robertson's recommended empirical parameters (alpha=0.50, beta=0.80) the delay indicator will be in error by 48%, the stops indicator will be in error by 147% and the performance indicator will be in error by 23%. One therefore concludes that significant errors in the signal performance characteristics can materialize by not calibrating Robertson's Recurrence Model.

For the predicted downstream cyclic flow profile which is based on a Calibrated Robertson's Model (alpha=0.40, beta=0.57) we note that RMSE index is decreased from 2.463

to 0.863, suggesting that a better fit is being obtained by calibration. A review of the three basic signal performance indicators for this case shows that with calibration the delay indicator will be in error by 24%; however, the improvement in the estimate of delay is in the order of 50% ((48-24)/48). Similarily for stops, the stop indicator will be in error by 31%; however, the improvement in the estimate of stops is in the order of 79% ((147-31)/147). Also, for the performance index, the indicator will be in error by 24%, however this makes no significant change (slight decrease in performance) as a result of calibration.

For the predicted downstream cyclic flow profile which is based on a <u>Calibrated Modified Robertson's Model</u> (alpha=0.40, beta=0.57), we note a further improvement in the RMSE index from 0.863 to 0.844, suggesting a even better fit can be obtained by providing an initialization equation. A review of the three basic signal performance indicators for this case shows that with modifications and calibration the delay indicator will still be in error by 16%, however this further improvement (calibrated modified Robertson's model in comparsion to a calibrated Robertson's model) in the estimate of delay is in the order of 33% ((24-16)/24). Similarily for stops, the stop indicator will still be in error by 28%; however, this further improvement in the estimate of stops is in the order of 10% ((31-28)/31). Also for the performance index, the performance indicator will still be in error by 16%; however, this further improvement

in the estimate of the performance index is in the order of 33% ((24-16)/31).

The last set of analysis is based on the predicted cyclic flow profile, which is based on an Modified Robertson's Model (alpha=0.50, beta=0.80). Generally speaking, in comparison to the Robertson's Model there is a consistent and significant improvement with respect to delays, stops, performance index and the RMSE statistics. However the improvements in performance are not comparable to those obtained by calibrating and modifing Robertson's model.

To obtain a comprehensive overview of all the analysis that was undertaken, a linear regression analysis of all predicted signal performance characteristics in comparison to the actual performance characteristics was undertaken in order to deduce, from a statistical perspective, the improvements relating to calibration and/or modifications to the model.

5.2.1 Delay Considerations

A regression analysis of the predicted minimal delay vs the actual minimal delay was undertaken for the data that was analyzed at all survey sites. The linear regression equation was of the form:

$$Y_d = a_1 + b_1 X_d$$

where Y_d = predicted minimal delay (seconds per vehicle),

X_d = actual minimal delay (seconds per vehicle), and

a_1, b_1 = calibrated coefficents.

For a perfect fit between predicted minimum delay vs actual minimum delay, the coefficient a_1 would assume a 0.00 value and the coefficient b_1 would assume a 1.00 value. Alternately, a good platoon dispersion model should approach these specific values for the coefficients.

The resulting regression equation for CASE I, CASE II and CASE III along with R-Square (R^2), and correlation coefficient (ρ) are listed below :

CASE I
Robertson's Model

$$Y_d = 0.607 + 0.791X_d \; ; \; R^2 = 0.92, \; \rho = 0.96$$

CASE II
Calibrated Robertson's Model

$$Y_d = 0.497 + 0.828X_d \; ; \; R^2 = 0.99, \; \rho = 0.99$$

CASE III
Calibrated Modified Robertson's Model

$$Y_d = 0.106 + 0.949X_d \; ; \; R^2 = 0.99, \; \rho = 0.99$$

The above equations indicate that, from a statistical perspective, the Calibrated Modified Robertson Model results in the best overall form of model for predicting minimal vehicular delay. Figure 5.1 illustrates all three delay regression lines along with the data points for CASE III. In reviewing this figure, it should be noted that all five of the six data points fall within the 95% confidence band. One

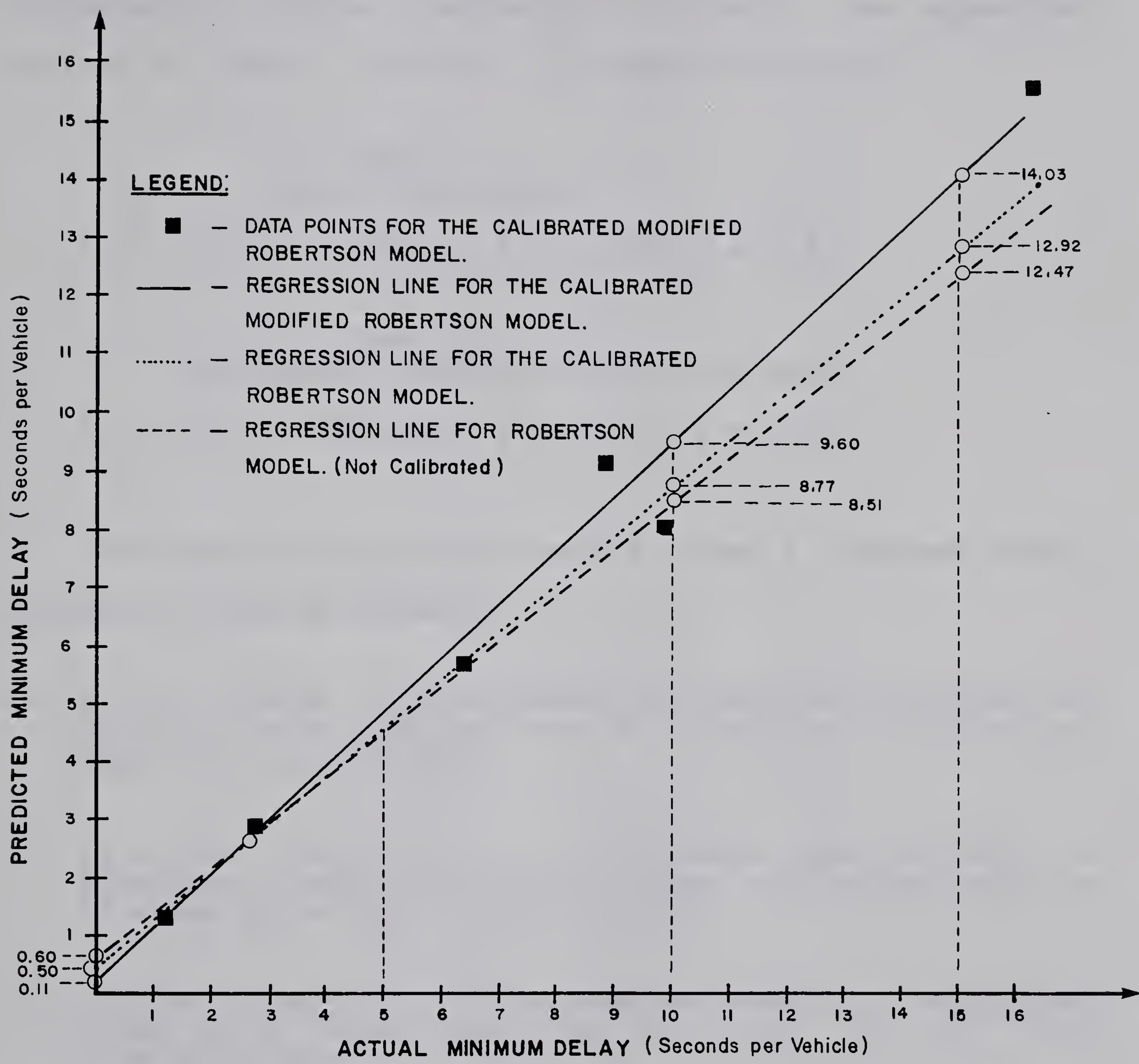


FIGURE 5.1

GOODNESS-OF-FIT FOR THREE FORMS OF ROBERTSON'S RECURRENCE MODEL FROM A MINIMUM DELAY PERSPECTIVE

data point misses this confidence band by approximately 0.2 seconds per vehicle.

For comparison purposes only, Lam's data [22] was subjected to similar regression analysis. The equations derived for CASE I and CASE III respectively were :

CASE I
Robertson's Model

$$Y_d = 1.091 + 0.791X_d \; ; \; R^2 = 0.69, \; \rho = 0.83$$

CASE III
Calibrated Modified Robertson's Model

$$Y_d = 0.997 + 0.912X_d \; ; \; R^2 = 0.79, \; \rho = 0.89$$

Principal conclusions reached from a mimimum delay perspective are as follows :

1. In the range of 0-5 seconds per vehicle the effect of calibration or modifications to Robertson's Recurrence Model is insignificant.

2. In the range of 5 - 15 seconds per vehicle, a Robertson's Model which has not been calibrated could be in error up to 20% ((15.00/12.47)).

3. In the range of 5-15 seconds per vehicle a Calibrated Robertson's Model could be in error up to 16% ((15.00/12.92)).

4. In the range of 5-15 seconds per vehicle a Calibrated Modified Robertson's Model could still be in error up to 7% ((15.00)/14.03)).

5. From the data collected and analyzed, no conclusion can be reached for delays greater than 15 seconds per vehicle. However, given the R^2 and ρ values, we can conclude that this equation is strong in its predictive power and that similar conclusion can be drawn for

larger delay values.

Figure 5.2 illustrates graphically the effect of calibration or modifications from a minimum delay perspective.

5.2.2 Stop Considerations

Undertaking a similar regression anlaysis for stops as was presented in Section 5.2.1, the following regression equations were obtained:

CASE I
Robertson's Model

$$Y_s = 0.105 + 0.433X_s \; ; \; R^2 = 0.74, \; \rho = 0.74$$

CASE II
Calibrated Robertson's Model

$$Y_s = 0.000 + 0.702X_s \; ; \; R^2 = 0.95, \; \rho = 0.97$$

CASE III
Calibrated Modified Robertson's Model

$$Y_s = 0.000 + 0.721X_s \; ; \; R^2 = 0.94, \; \rho = 0.96$$

Based on the above equations, we conclude that, from a statistical perspective the Calibrated Modified Robertson Model results in the best overall form of the model for predicting minimum vehicular stops. Figure 5.3 illustrates all three stop regression lines along with the data points for CASE III. In reviewing this figure it should be noted that all six data points fall within the 95% confidence bands.

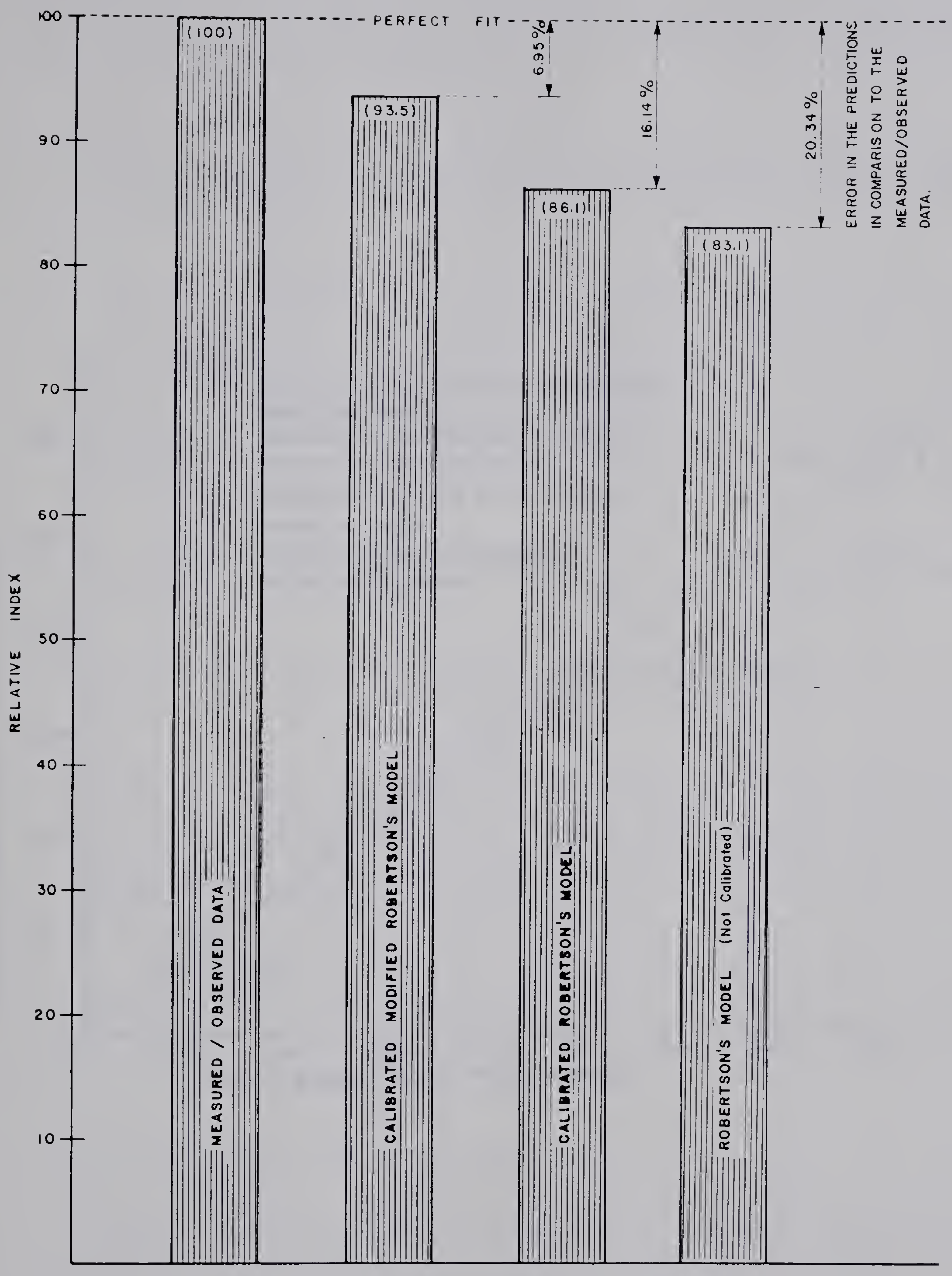


FIGURE 5.2

EFFECT OF CALIBRATION OR MODIFICATIONS FROM A MINIMUM DELAY PERSPECTIVE

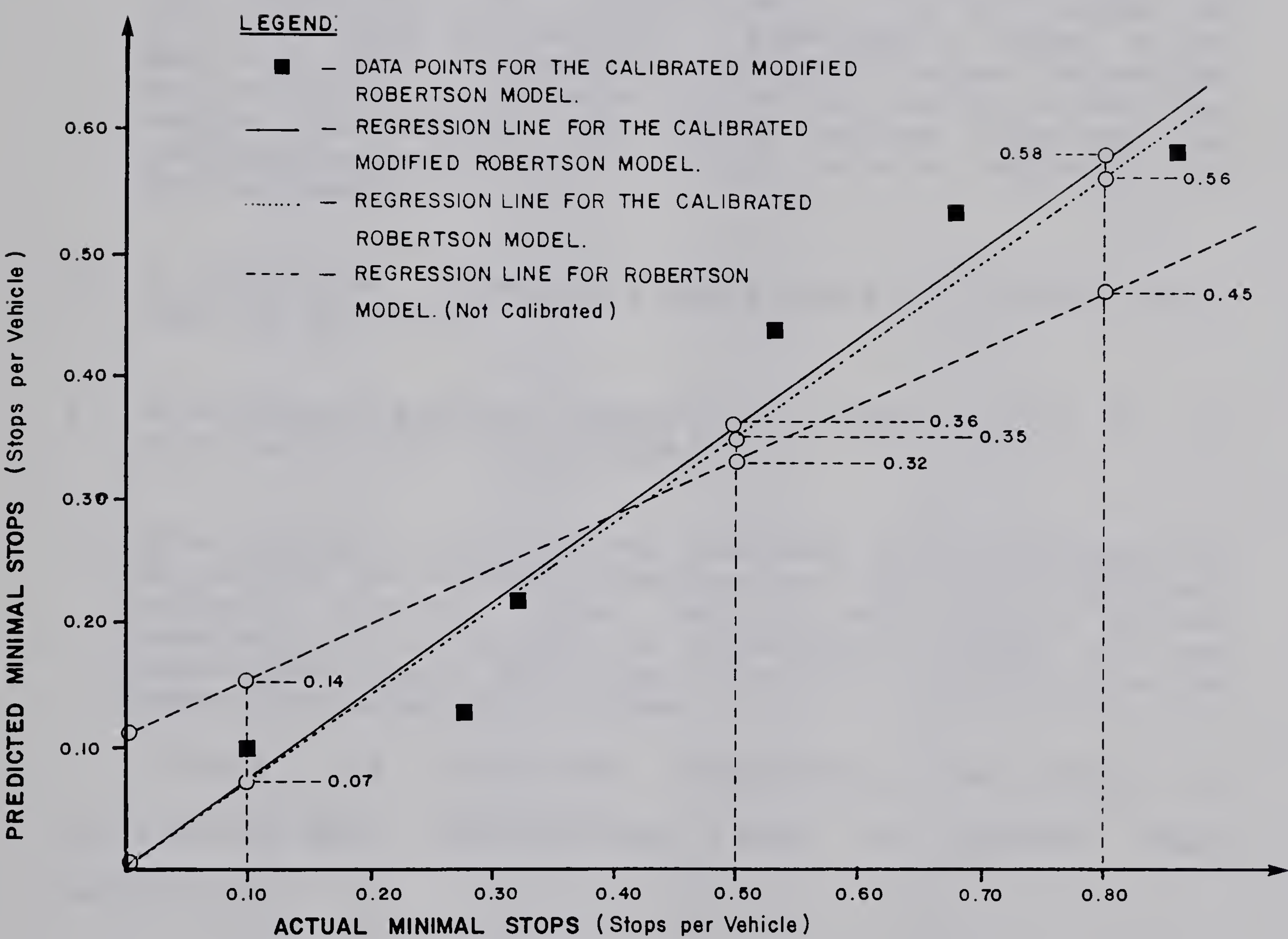


FIGURE 5.3

GOODNESS-OF-FIT FOR THREE FORMS OF ROBERTSON'S RECURRENCE MODEL FROM A MINIMUM STOPS PERSPECTIVE

Principal conclusions that are reached from a minimum stop perspective are as follows :

1. In the range of 0.35-0.45 stops per vehicle the effect of calibration of modifications to Robertson's model is insignificant.

2. In the range of 0.45-0.80 stops per vehicle a Robertson's Model which has not been calibrated could be in error by 78% ((0.80)/0.45)). In the range of 0.00-0.45 stops per vehicle, a Robertson's Model which has not been calibrated could have larger percentage errors; however, the magnitude of this error in absolute values is smaller. The former errors represent an underestimation while the later errors represent an overestimation.

3. A Calibrated Robertson's Model could be in error up to 43% ((0.80/0.56)).

4. A Calibrated Modified Robertson's Model could be in error up to 38% ((0.80)/0.58)).

5. From the data collected and analyzed, no conclusions can be reached for minimum stops greater than 0.85 stops per vehicle. However, given the R^2 and ρ values, we can conclude that this equation is strong in terms of its predictive power and that similar conclusions can be drawn for larger stop values.

Figure 5.4 illustrates graphically the effect of calibration and modifications from a minimum stop perspective.

5.2.3 Performance Index Considerations

Again, a similar regression analysis for the performance index as was presented in Section 5.2.1 and Section 5.2.2 resulted in the following regression equations:

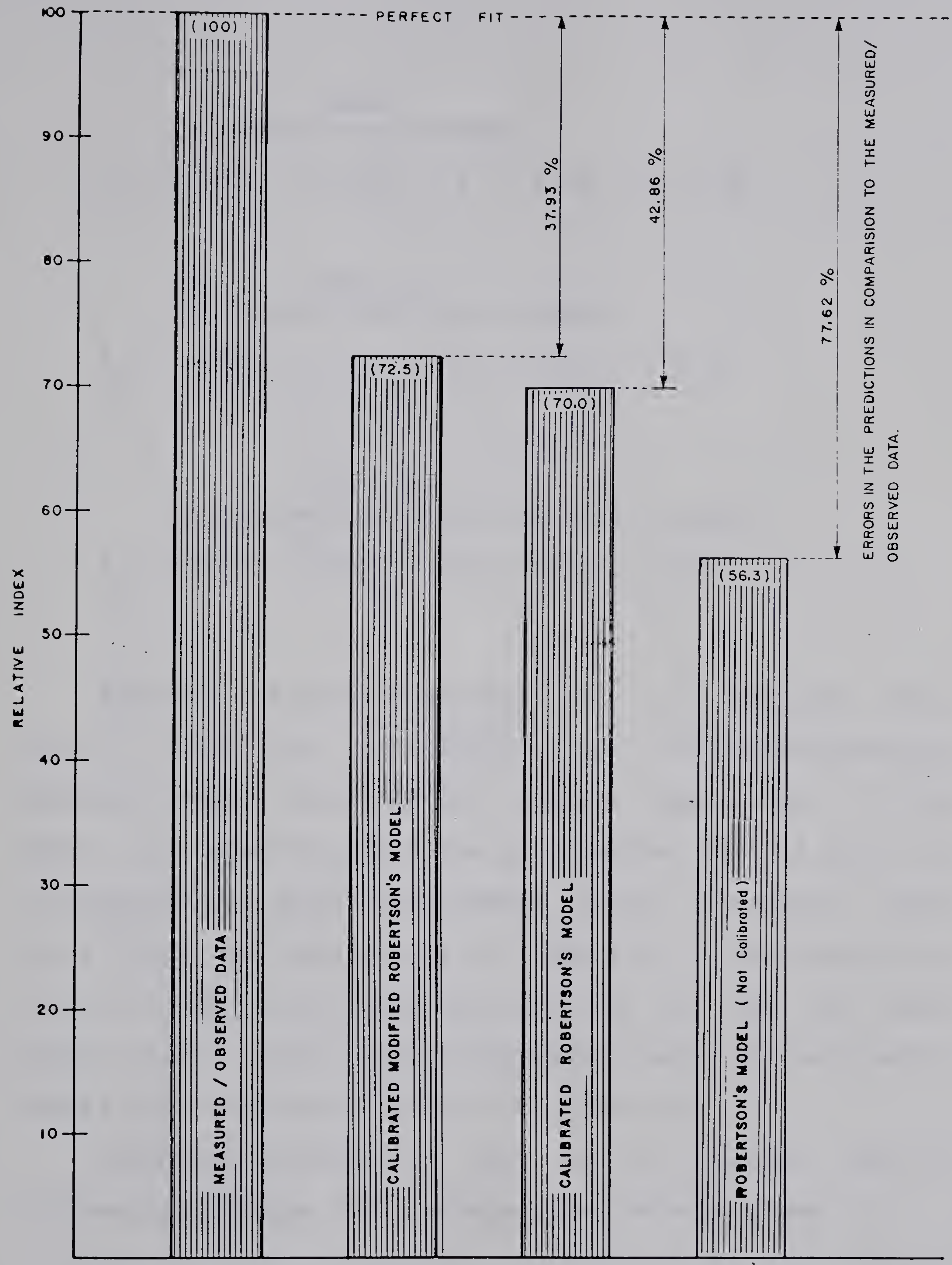


FIGURE 5.4

EFFECT OF CALIBRATION OR MODIFICATIONS FROM A MINIMUM STOPS PERSPECTIVE

CASE I
Robertson's Model

$$Y_p = 0.041 + 0.877X_p \;;\; R^2 = 0.99,\ \rho = 0.99$$

CASE II
Calibrated Robertson's Model

$$Y_p = 0.120 + 0.861X_p \;;\; R^2 = 0.99,\ \rho = 0.99$$

CASE III
Calibrated Modified Robertson's Model

$$Y_p = 0.003 + 0.963X_p \;;\; R^2 = 0.99,\ \rho = 0.99$$

Based on the above equations, we can conclude, that from a statistical perspective, the Calibrated Modified Robertson Model results in the overall best form of the model for predicting minimum performance index. Figure 5.5 illustrates all three performance index regression lines along with the data points for CASE III. In reviewing this figure, it should be noted that all five of the six data points fall within the 95% confidence band. One data point misses this confidence interval by only 0.09.

Principle conclusions that can be reached from a minimum performance index perspective are as follows :

1. Robertson's Model which has not been calibrated could be in error up to 14% ((10.00)/8.82)).

2. A Calibrated Robertson's Model could be in error up to 15% ((10.00)/8.73). The differences between a calibrated

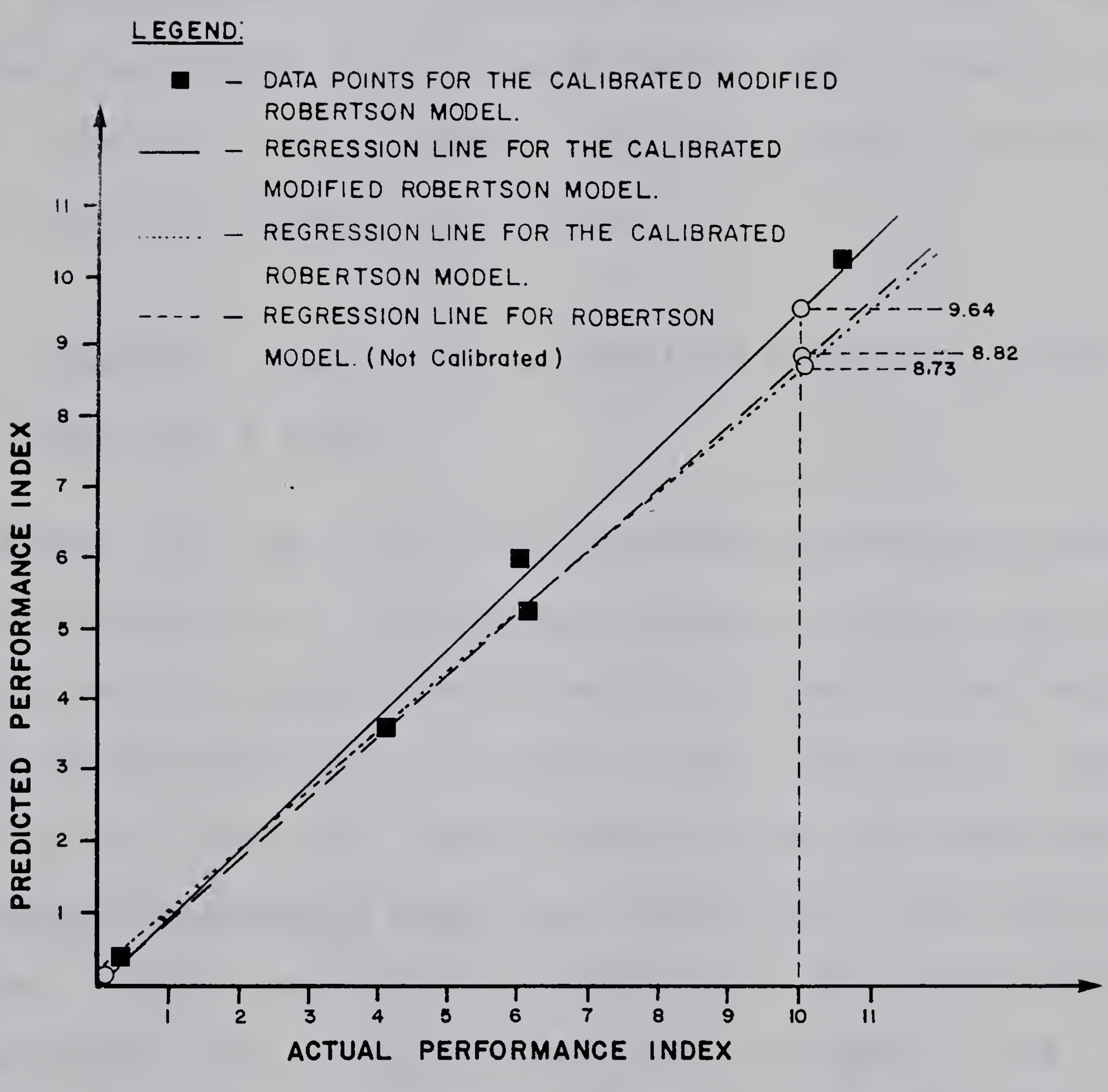


FIGURE 5.5

GOODNESS-OF-FIT FOR THREE FORMS OF ROBERTSON'S RECURRENCE MODEL FROM A MINIMUM PERFORMANCE INDEX PERSPECTIVE

and a non-calibrated Robertson's Model appear to be insignificant.

3. A Calibrated Modified Roberton's Model could be in error up to by 4% ((10.00)/96.4).

Figure 5.6 illustrates graphically the effect of calibration and modifications from a minimum performance index perspective. It should be noted that the use of a different weighting factor (performance index equals total delay (uniform plus random) plus four times the number of stops) may alter these conclusions.

5.2.4 Overall Impact Of Calibration And Modifications To Robertson's Model

Based on the previously discussed regression analysis we conclude that the Calibrated Modified Robertson's Model can improve the predictive estimates of the minimum value of delays by 13% (93.5/83.1), stops by 29% (72.5/56.3), and the performance index by 9% (96.4/88.2). On the other hand, a Calibrated Robertson's Model can improve on the previously defined signal performance indicators by 4%, 24% and 11% respectively. A similar regression analysis was also duplicated for the complete range of offsets (all offset values including the minimum value) for delays and stops at each intersection surveyed and analyzed. Similar conclusive results were obtained. Calibration of the individual value of alpha and beta along with some modifications to the model, will consistently improve the prediction of signal

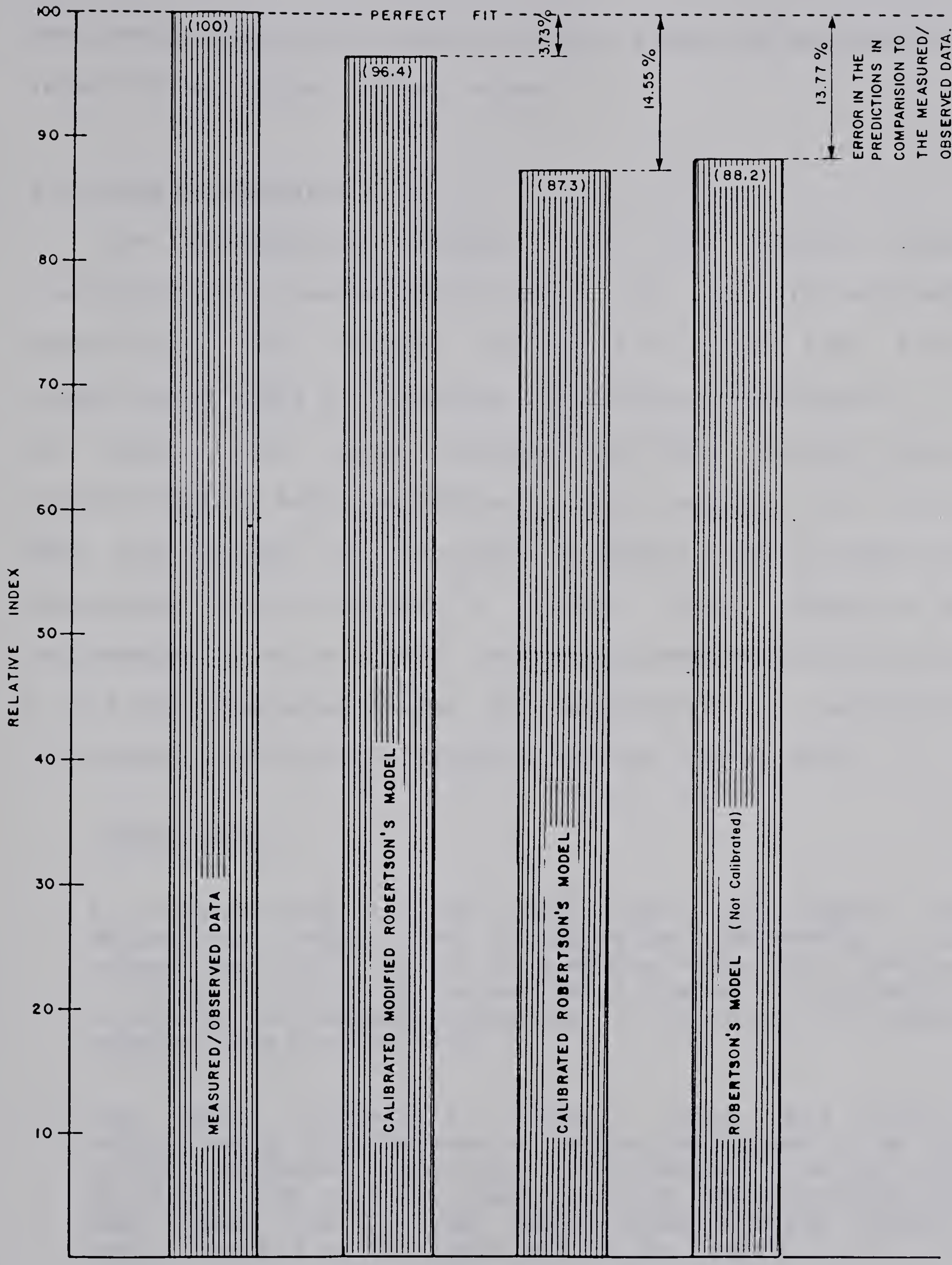


FIGURE 5.6

EFFECT OF CALIBRATION OR MODIFICATIONS FROM A MINIMUM PERFORMANCE INDEX PERSPECTIVE

performance indicators such as delays, stops and performance index for any signal offset value.

5.3 Design Guidelines

The calibration results for the survey sites investigated are summarized in Table 5.7, and illustrated graphically in Figures 5.7 to 5.12. For each site investigated Table 5.7 presents the following information : the alpha value which minimized the RMSE, the beta value which minimized both the RMSE and the rounding off error when the product of beta and the travel time in steps is calculated, the resulting K value (note Robertson's recommended K value is 40), the calculated smoothing factor F, and the associated degree of saturation. In reviewing this table the following trends have been identified :

ALPHA VALUE

1. At intersections (summer data) where the degree of saturation ranges from a high value (104 Avenue - 116 Street, Week No. 1 and 2) to a medium value (111 Avenue - 116 Street) to a low value (102 A Avenue - 97 Street), we note a corresponding decrease in the value of alpha ranging from 0.45 to 0.15

2. The basic difference between summer and winter conditions at high degrees of saturation appear to be a slight decrease in the alpha value (Summer 0.45 to 0.40 vs Winter 0.40 to 0.30). Severe winter conditions are at the higher end of the range while average winter conditions are at the lower end of the range.

TABLE 5.7

CALIBRATED PARAMETERS FOR THE SURVEY SITES INVESTIGATED DURING THE A.M. PEAK HOUR

Survey Location	Alpha	Beta	Travel Time (T) Seconds	Travel Time (t) Steps	K Value $\frac{(\text{Alpha} * \text{Beta})}{100}$	Smoothing Factor (F)	Degree Of Saturation
104 Avenue - 116 Street (Severe Winter Conditions)	0.40	0.570	14.04	7.02	22.8	0.385	92%
104 Avenue - 116 Street (Average Winter Conditions Further Downstream)	0.30	0.725	8.24	4.140	21.8	0.525	88%
104 Avenue - 116 Street (Week No. 1)	0.45	0.811	9.87	4.935	36.5	0.357	96%
104 Avenue - 116 Street (Week No. 2)	0.40	0.811	9.87	4.935	32.4	0.385	92%
111 Avenue - 116 Street	0.35	0.722	11.08	5.54	25.3	0.417	44%
102 A Avenue - 97 Street	0.15	0.627	12.76	6.38	9.4	0.625	29%

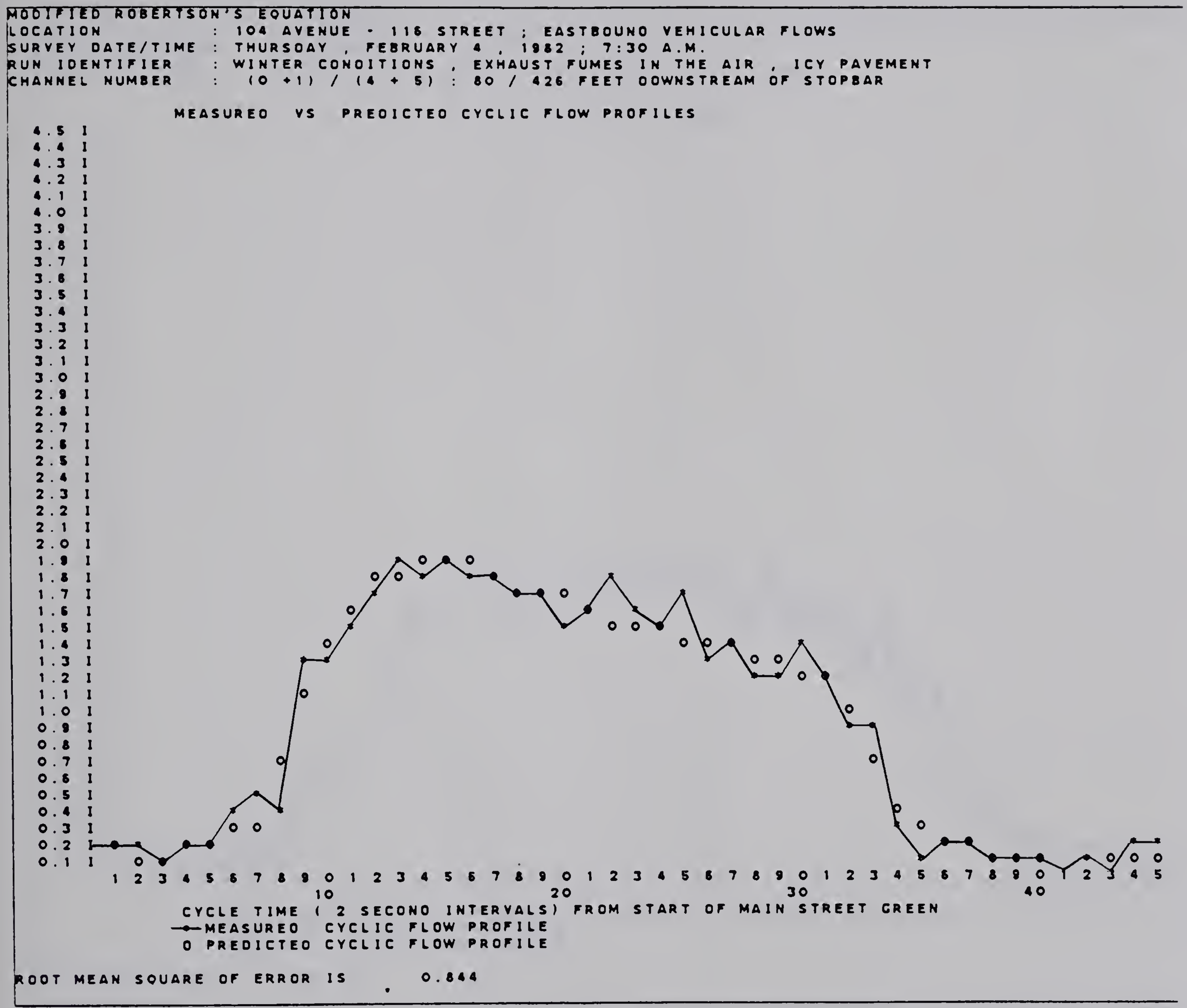


FIGURE 5.7

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 104 AVENUE - 116 STREET (SEVERE WINTER CONDITIONS) WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.40, BETA = 0.570

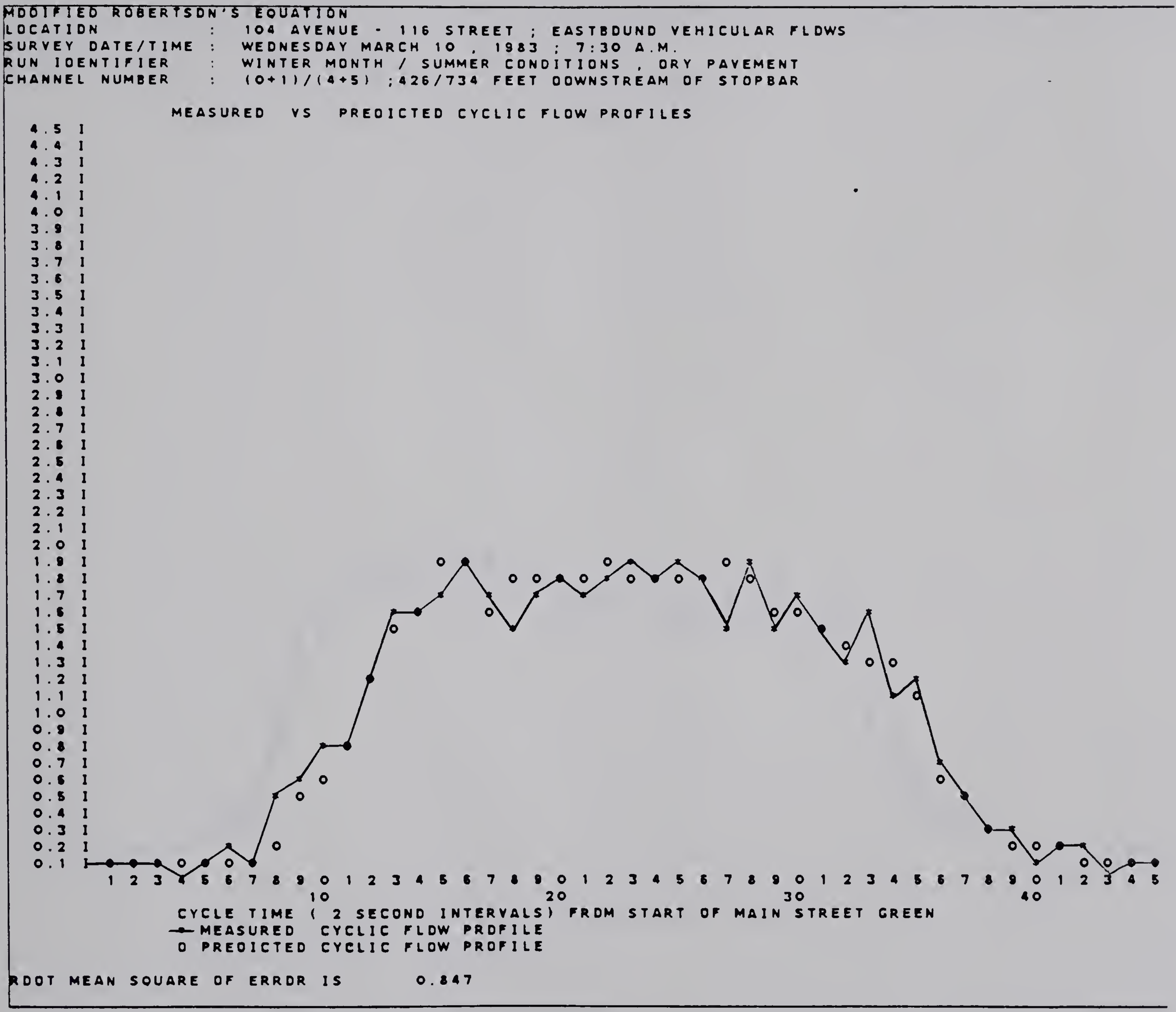


FIGURE 5.8

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 104 AVENUE - 116 STREET (AVERAGE WINTER CONDITIONS, FURTHER DOWNSTREAM) WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.30, BETA = 0.725

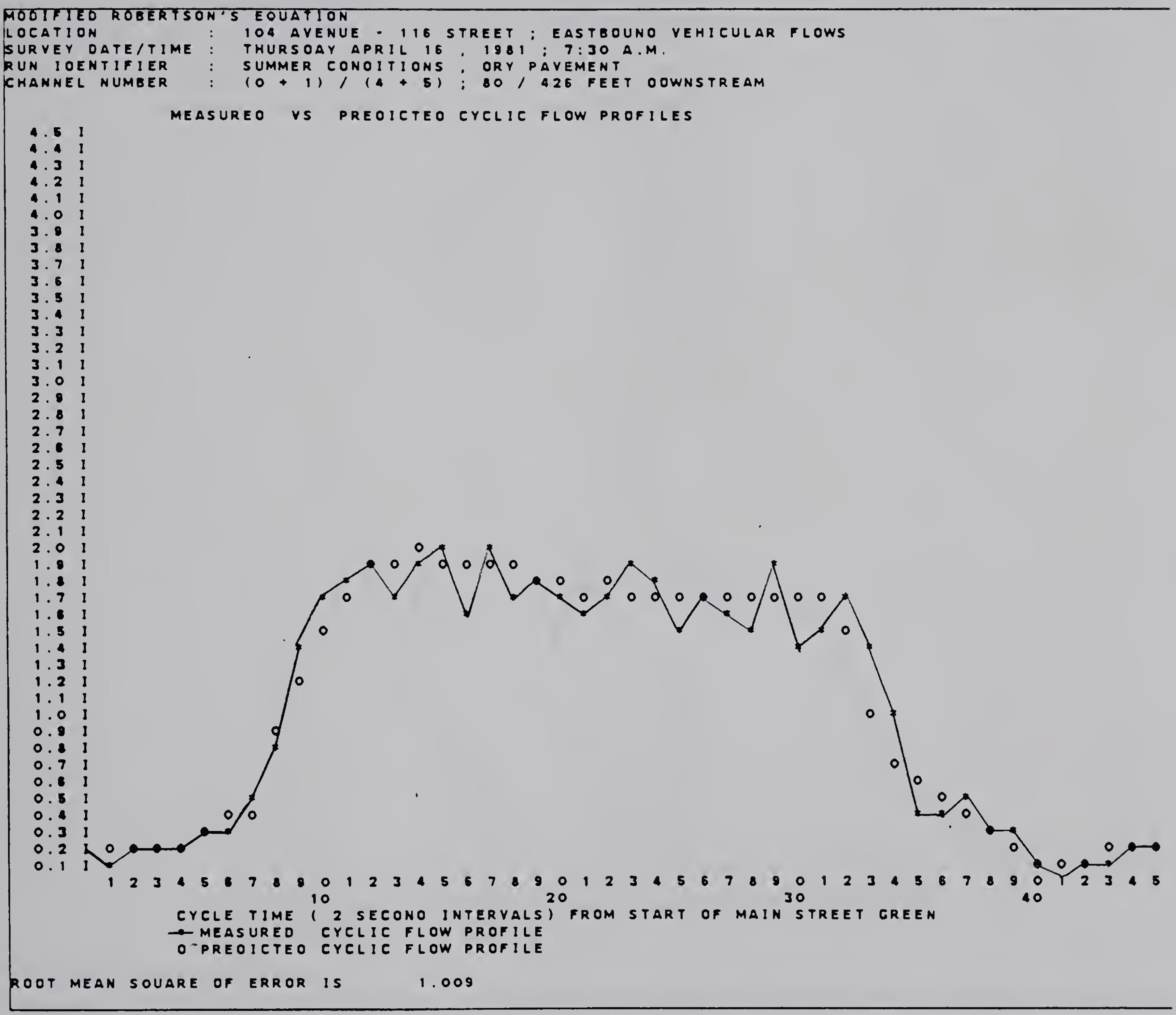


FIGURE 5.9

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 104 AVENUE - 116 STREET (WEEK NO. 1) WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.45, BETA = 0.811

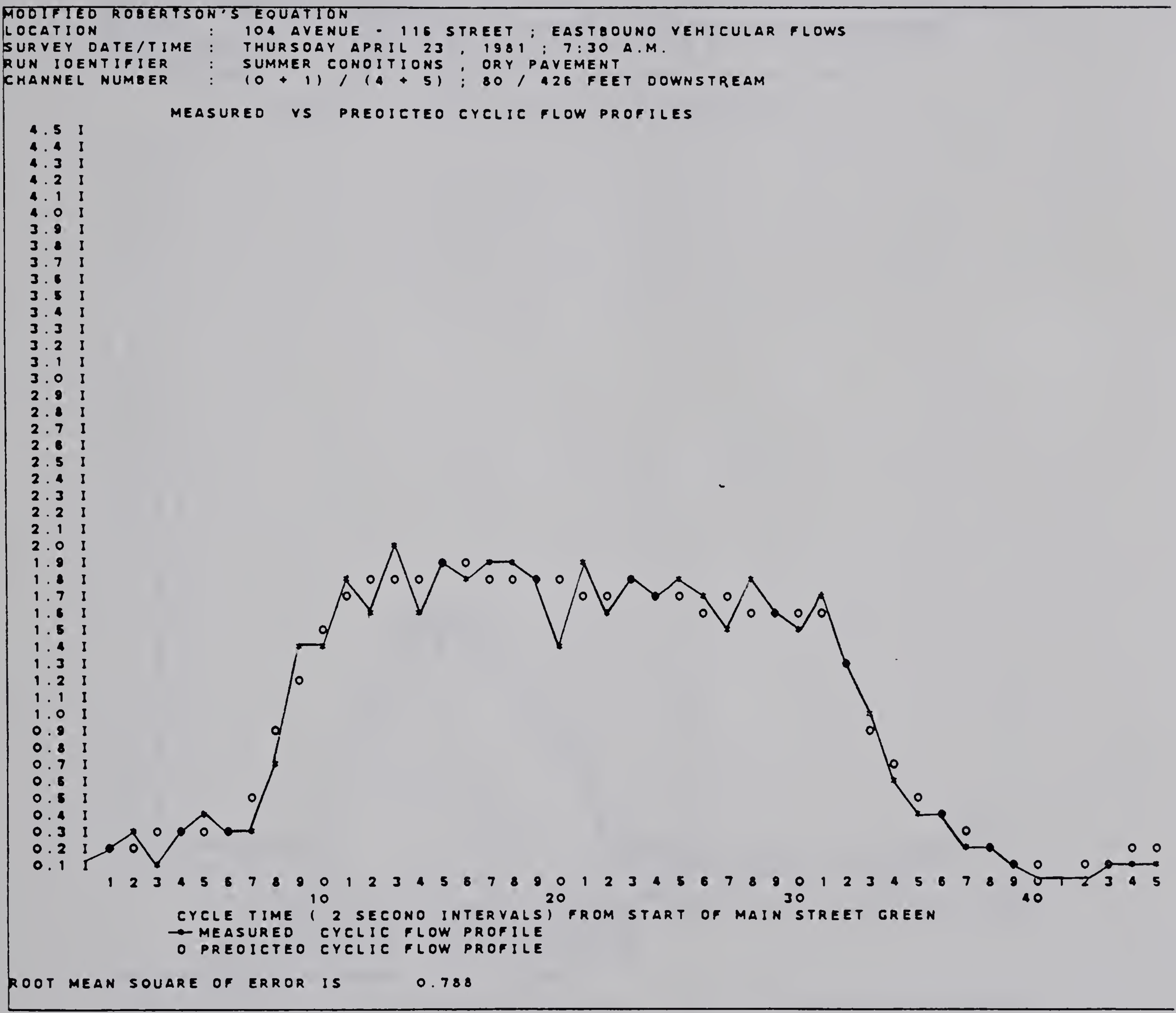


FIGURE 5.10

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 104 AVENUE - 116 STREET (WEEK NO. 2) WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.40, BETA = 0.811

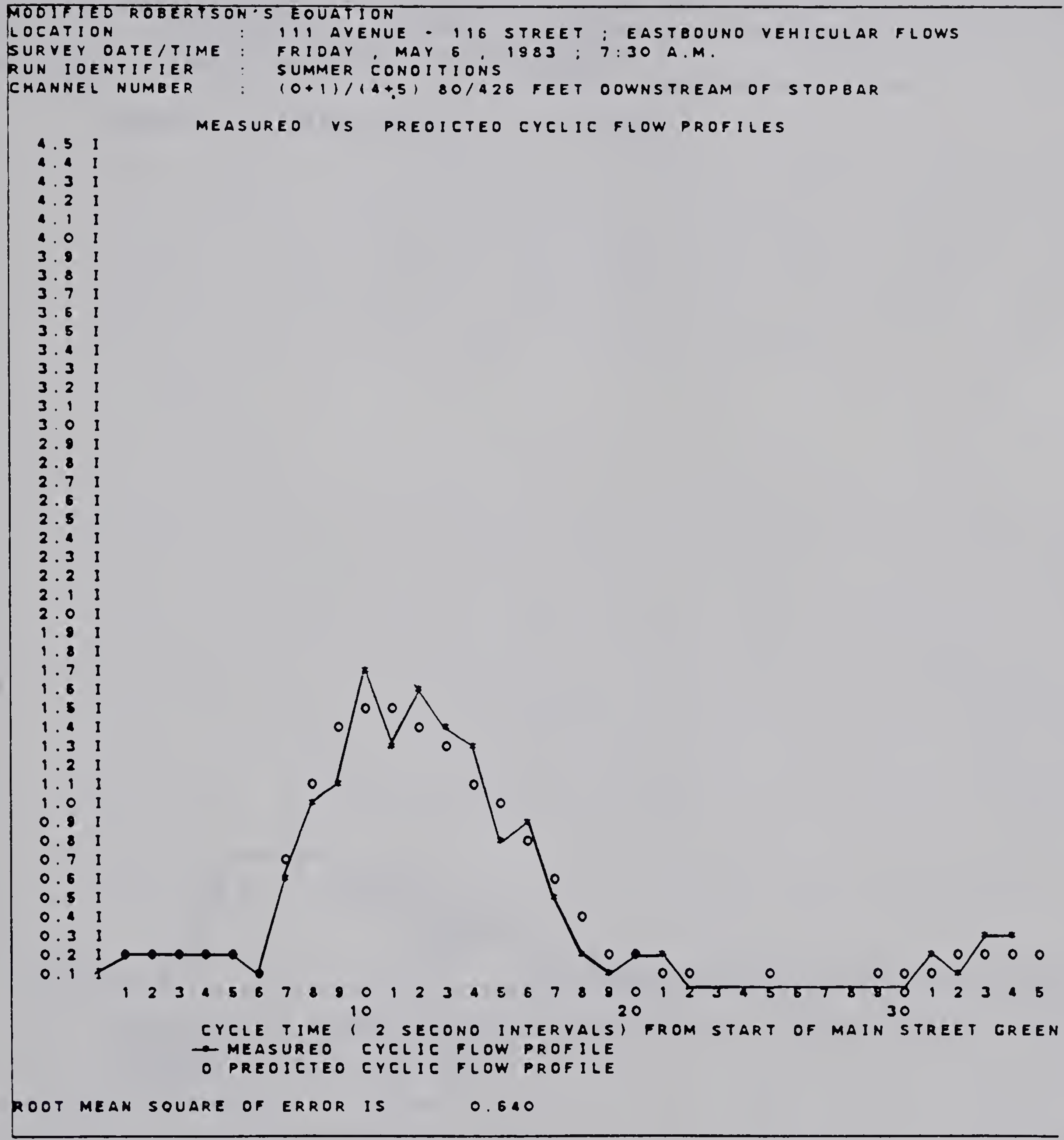
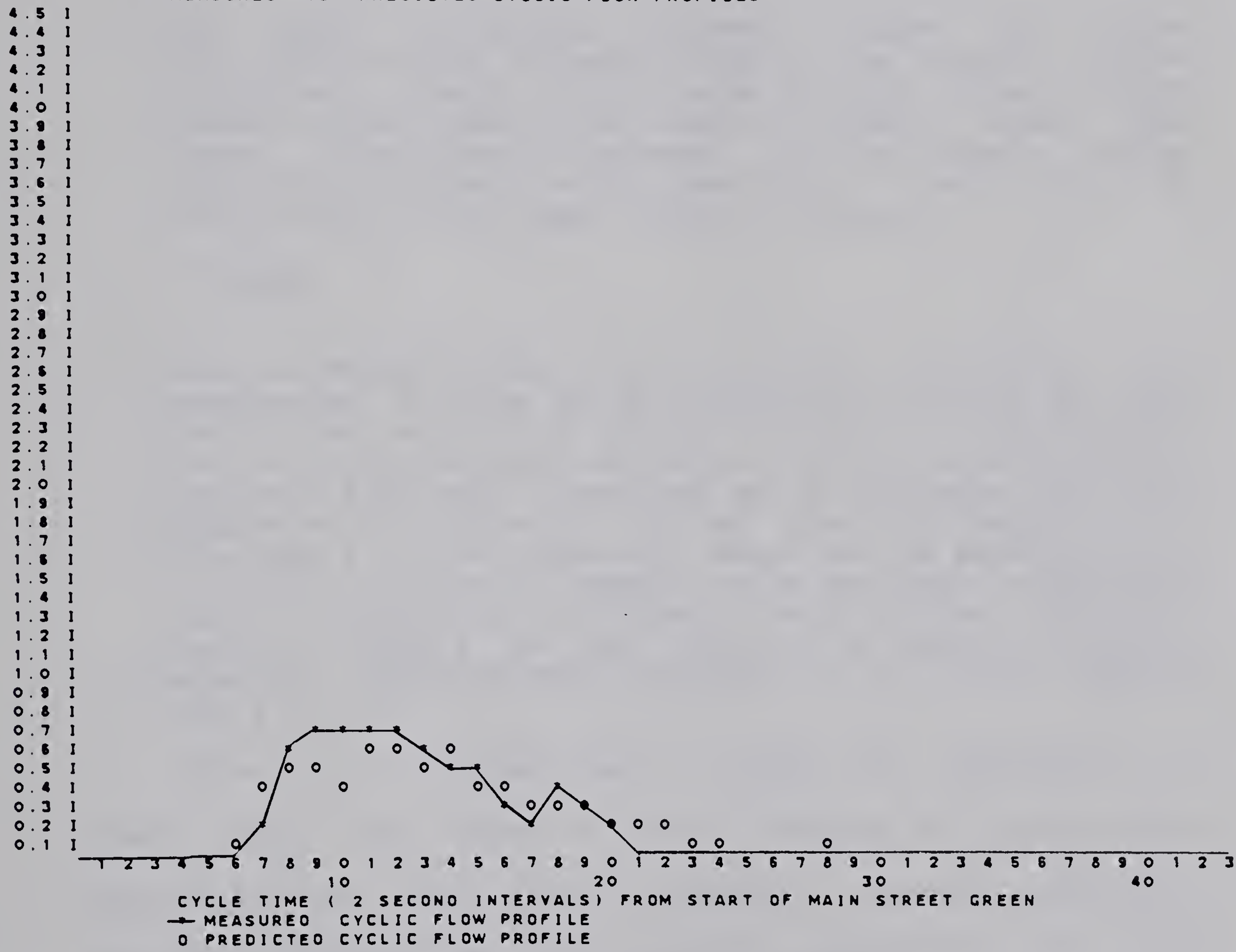


FIGURE 5.11

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 111 AVENUE - 116 STREET WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.35, BETA = 0.722

MODIFIED ROBERTSON'S EQUATION
LOCATION : 102A AVENUE - 97 STREET ;SOUTHBOUND VEHICULAR FLOWS
SURVEY DATE/TIME : FRIDAY , OCTOBER 14 , 1983 ; 7:25 A.M.
RUN IDENTIFIER : SUMMER CONDITIONS , DRY PAVEMENT ....
CHANNEL NUMBER : (0 + 1) / (4 + 5) : 70 / 345 FEET DOWNSTREAM OF STOPBAR

MEASURED VS PREDICTED CYCLIC FLOW PROFILES



ROOT MEAN SQUARE OF ERROR IS 0.620

FIGURE 5.12

COMPARISON OF MEASURED (*) VS PREDICTED (O) CYCLIC FLOW PROFILE FOR 102 A AVENUE - 97 STREET WITH THE FOLLOWING CALIBRATED ROBERTSON'S PARAMETERS : ALPHA = 0.15, BETA = 0.627

BETA VALUE

1. At intersections (summer data) where the degree of saturation ranges from a high value (104 Avenue - 116 Street Week No. 1 and 2) to a medium value (111 Avenue - 116 Street) to a low value (102 A Avenue - 97 Street), we note a corresponding decrease in the value of beta ranging from 0.811 to 0.627.

2. The basic difference between summer and winter conditions at high degrees of saturation appears to have significant impact on the range of beta values. During typical winter conditions, beta is slightly lower than summer conditions; however, during severe winter conditions, the beta value observed is lower than the lowest value of all beta values in summer.

K VALUE

1. When the TRANSYT model was initially introduced, the recommended K value was 40 (alpha=0.50, beta=0.80). The most current version of TRANSYT, Version 8, recommends a K value of 35 (alpha=0.44, beta=0.80) in response to the available published literature which indicates that more dispersion is actually taking place in comparsion to Robertson's orignal research. Based on the Edmonton data a K value of 35 still appears to be too high, suggesting that in comparsion to other traffic flow patterns, Edmonton drivers do not exhibit a compact platoon behaviour. Edmonton data results in a K value ranging from 9 to 37.

Robertson ([13])stated that it would be reasonable to expect that the value of F be a function of various site factors such as road width, gradient, parking activity, opposing vehiclar flow level, traffic composition, etc., and not simply the average journey time (T). Robertson when making this statement, was in fact indicating that the alpha value would only be a function of these characteristics since the basic input into the TRANSYT model (Card Type 31 - Link/Node Relation) permits the user to input K. A closer examination indicates that through K, the user could only

change alpha and not the value of beta. The value of beta = 0.80 could not be changed by the user.

This Researcher, with the same reasoning, suggests that it would also seem reasonable to expect that βT, which in effect is the average arrival time of leading vehicles in the platoon, should depend to some extent on these same site factors (perhaps more importantly to ambient weather conditions) and not simply 0.8 of the average journey time (T). The Reseracher further suggests, that the amount of platoon dispersion (which is a function of alpha, beta and travel time) could be correlated with the degree of saturation at the upstream signalized intersection. As degree of saturation increases, the freedom to maneouvre decreases, and hence we would expect that the amount of dispersion to decrease.

The Edmonton data as presented in Table 5.7 provides some conformation that alpha and beta could be a function of site characteristics. It appears that degrees of saturation (ratio of volume to capacity) are perhaps a better indicator of the most appropriate value of alpha and beta for design/analysis purposes. For this reason, it is recommended that the information presented in Table 5.7 be used as a design guideline. The Edmonton data also suggests that when winter signal timing plans are being prepared, estimates of winter saturation flow rates ([35]) and winter travel times alone, are not adequate inputs for signal design. Different emperical values of alpha and beta in comparsion to summer

conditions are also required. Based on a weighted average (weighted in proportion to the number of cycles of data collected) the following average empirical parameters may also be used for general design purposes :

1. AVERAGE EMPIRICAL PARAMETERS FOR USE IN ROBERTSON'S RECURRENCE MODEL DURING SUMMER CONDITIONS IN EDMONTON, ALBERTA, CANADA : ALPHA = 0.34, BETA = 0.75.

2. AVERAGE EMPIRICAL PARAMETERS FOR USE IN ROBERTSON'S RECURRENCE MODEL DURING WINTER CONDITIONS I EDMONTON, ALBERTA, CANADA : ALPHA = 0.34, BETA = 0.66.

5.4 Practical Applications

Knowledge of platoon dispersion has many applications[26,22] apart from its main use in deriving accurate downstream arrival patterns for signal offset design and evaluation through the computation of delay, stops, and performance index relationships. The height of the peaks and trough of the cyclic flow profile, and their shape and location in time, provide the transportation engineer with a great deal of addition information as outlined below:

1. *Average flow.* The area of the cyclic flow profile is proportional to the average flow. When inspecting several such graphs, sites with heavy flows are easily identified.

2. *Major/minor road inflows.* If the green times along with the signal phasing sequence at the upstream intersection are known, then the major and minor inflow rates can be checked by observing the average flow during the appropriate part of the cycle.

3. *Benefits of signal co-ordination* The variation in the

height of the cyclic flow profile is a measure of the need for signal co-ordination. If the height is nearly constant during the cycle, then there is no benefit to be gained by co-ordinating the upstream and downstream signals. This is true even when the average flow is large. Conversely, a cyclic flow profile with a pronounced peak corresponding to a well defined platoon presents a strong potential for good signal co-ordination.

4. *Saturation flow.* Saturation flow is defined as the rate of flow across the stopline from a queue, but it can be deduced from measurements at a site just downstream of an intersection. During some part of the signal cycle (during that portion of the cycle when a stationary queue exists), traffic will discharge at the saturation rate. With knowledge of upstream signal timings, the magnitude, duration, and effect of length of green on saturation flow can be checked from the cyclic flow profile.

5. *Spare capacity.* The part of the entire green time during which the actual flow is below saturation flow levels represents the space capacity at the upstream signal approach.

6. *Platoon speed and dispersion.* The location in time of the peaks and troughs of the cyclic flow profile will depend on the upstream signal timing, the time of traffic platoons arriving at the upstream signal, and the distance between the up-stream signals and the site at which the cyclic flow profile is measured.

7. *Signal start and end lags.* During a fully saturated signal phase the observed cyclic flow profile can be used to calculate signal start and end legs ([31]). A start lag is the initial hesitation following the display of green prior to traffic discharging at a constant rate. A end lag is the time when the display of amber results in traffic discharging at a rate which reduces sharply to zero.

8. *Incident detection.* The shape of the cyclic flow profile will alter during the normal day. It may alter more rapidly if there is an accident or other incident at an upstream signal; change in cyclic flow profile shape may be a sensitive indicator of traffic incidents. (This specific type of research is presently being undertaken jointly by the Civil Engineering and Electrical

Engineering Departments at the University Of Alberta.)([28]) The computing program BRAD#+*PLOTLIB permits the user to view graphically the variations (see Figure 3.4) in the cyclic flow profile from one cycle to the next.

9. *Signal setting by TRANSYT.* Cyclic flow profile traffic analysis is likely to be particularly useful when the TRANSYT method of optimizing fixed-time plant has been employed to derive the signal timings. The TRANSYT computer program prints a cyclic flow profile graph for each part of the network. Comparision of predicted and measured cyclic flow profile's can be inspected and quickly reveal any serious differences between theory and practice. Corrections can then be made to the data fed to TRANSYT and an improved standard of traffic control will result.

10. *International exchange standard.* In the view of Robertson, there is considerable merit in using cyclic flow profiles to exchange information on the behaviour of urban road traffic in different cities and countries. Comparison between measured and predicted cyclic flow profiles confirms the accuracy of the existing methods and highlights areas for further research and development. For example, advisory speed are displayed to motorists in some co-ordinated signal systems. To be effective such signs would be expected to alter the downstream CFP shape. It would be interesting to establish whether they do.

    As a further example, it may be possible as this reseach has indicated on a preliminary level to establish relationships between weather conditions and the cyclic flow profile shape. An icy road not only affects saturation flow rates, but also journey times and platoon dispersion factors. At the present time, little is known of the magnitudes of these effects.

    Teply ([32]) provided a definition of a framework for evaluation of the applicability of foreign experience in traffic management, considering behavioural differences of drivers and pedestrians in various countries. The element of individual and collective behaviour as measured by traffic platoons (dispersions, headways and spacing, headway stability) either constitutes or significantly influences traffic behaviour. Traffic platoons, therefore is one measure to report deviations from the accepted level.

11. *Preferential street treatment.* To provide guidelines in the traditional preferential street treatment of co-ordination design. Streets with compact platoon

movements can be favoured over streets with considerable platoon dispersion.

12. *Ranking of network links.* To aid in the ranking of network links in order of their importance in terms of effectiveness of co-ordination. Link ranking is important in off-line signal control strategies such as SIGRID (SIgnal GRid Design) Program ([37]) where links are weighted by their importance before optimization.

13. *Need for coordination.* To provide an indication of co-ordination needs. For example, links without platoon movements can be excluded from the network description to save computing time.

14. *Network condensation.* To aid in network condensation for easier analysis. For example, the Combination Program([23]) which is only applicable to certain complex networks can be condensed to suit the program by the deletion of links with relatively ill-defined platoon structures.

15. *Understanding the flow characteristics at a midblock access road between two signalized intersections.* Estimating the shape of a cyclic flow profile flow for a two-way road at a midblock access capacity analysis. If the platoon from both directions of flow cross the midblock access road at the same time, adequate gaps for the minor street flow may be created. On the other hand, staggering the arrival of both platoons may imply that a capacity deficiency may materialize at the access road.

6. SUMMARY

6.1 Conclusion

The results of this research have shown that the dispersion of traffic platoons between fixed-time signalized intersections can be simulated quite accurately using Robertson's Recurrence Model. In the absense of locally available information, Robertson's recommended empirical parameters of alpha=0.50 and beta=0.80 or more recently alpha=0.44 and beta=0.80 represent good <u>starting approximations</u>. Under circumstances such as winter weather conditions and degrees of saturation less than 90% on the street system, Robertson's recommended parameters will be in error. The effect of these errors has a signficant effect on the accuracy of signal offset design which is measured by such indicators as delays, stops and performance index. Significant improvements can be made by calibrating the two individual empirical parameters and introducing an initialization equation (Modified Robertson's Recurrence Model) to start the prediction process. Specifically, the following errors in signal performance are noted :

<u>MINIMUM VEHICULAR DELAY</u>

1. A Robertson's Recurrence Model which has <u>not</u> been calibrated to local conditions could be in error up to 20%.

2. A Robertson's Recurrence Model which has been calibrated to local conditions could be in error up to 16%.

3. A Modified Robertson's Recurrence Model which has been calibrated to local conditions could still be in error up to 7%.

MINIMUM VEHICULAR STOPS

1. A Robertson's Recurrence Model which has not been calibrated to local conditions could be in error up to 78%.

2. A Robertson's Recurrence Model which has been calibrated to local conditions could be in error up to 43%.

3. A Modified Robertson's Recurrence Model which has not been calibrated to local conditions could still be in error up to 38%.

MINIMUM PERFORMANCE INDEX

1. A Robertson's Recurrence Model which has not been calibrated to local conditions could be in error up to 14%.

2. A Robertson's Recurrence Model which has been calibrated to local conditions could be in error by 15%.

3. A Modified Robertson's Recurrence Model which has been calibrated to local conditions could still be in error up to 4%.

Based on the analysis of six survey sites, the Researcher concludes that both calibration and modifications to Robertson's Recurrence Model can reduce the error in prediction of delay by a factor of 3, error in prediction of stops by a factor of 2, and error in prediction of the performance index by a factor of 3.5. These conclusions have

significant implications in the following areas :

1. The design of signal timings along a one way arterial where the objective is to minimize the number of stops. An error in the estimate of the number of stops by a factor of 2 will result in sub-optimal signal design.

2. The design of network timings in the TRANSYT model is based on an optimization procedure. The offsets between signals are adjusted by a "hill-climbing" optimization procedure to minimize an index of performance. The index of performance is a weighted combination of delay and stops. Errors in delay estimates and stops estimates, when combined into a performance index generally will result on average in a larger error. Optimization of an index which is in error will result in sub-optimal network timing design.

6.2 Recommendations

To improve on the prediction of several signal performance indicators such as delay, stops and performance index the following recommendations are put forth:

1. Robertson's Recurrence Model in the TRANSYT method of signal design should be modified by introducing an initialization equation to the prediction process.

2. Basic imput into the TRANSYT method of signal design should be modified to allow the individual empirical parameters of alpha and beta as input parameters.

3. In the absence of data for a specific signal network, the following empirical parameters should be used in Edmonton, Alberta (Canada):

    <u>SUMMER CONDITIONS</u> : Alpha = 0.34, Beta = 0.75
    <u>WINTER CONDITIONS</u> : Alpha = 0.34, Beta = 0.66

4. The Micro-computer Cyclic Flow Profile Monitor has been demonstrated to be an excellent tool for collecting

real-time traffic platoon data. It is estimated that with two technical support staff and two-three hours of total survey time (setting up of the machine, survey itself, dis-assembly of survey equipment), approximately one-half to one hour of data can be collected to a resolution of one-tenth of a second. With the software packages (BRAD#+*PLOTLID, GORDONV1, GORDONV2 and DSPI) that were specifically designed for this research, it is estimated that one to one-half hour of support staff time would be required to obtain the most appropriate value of alpha or beta. The City Of Edmonton traffic signal design/analysis team is, therefore, encouraged to use the data collection technique and the various software programs developed for this research. <u>This on going process of data collection and analysis at selective roadway links is needed to confirm that the predictions by TRANSYT reflect the actual field conditions</u>.

6.3 Further Research Needs

The following further research needs have been identified:

1. The initialization equation introduced into the Modified Recurrence Model was based on the assumption of steady-state conditions. Additional research into the validity of this assumption is required especially when it is observed during peak time periods, that time varying demands occur.

2. The analysis of the data base presented, indicates that the individual value of the alpha and beta parameters is influenced by weather conditions (summer vs winter) and degree of saturation. Specifically, it was observed that, with the summer data, a consistent trend (as degree of saturation increased the individual value of alpha and beta increased) appeared. More winter and summer data could be instrumental in defining more refined design/ analysis guidelines in the use of the two empirical parameters.

3. More study is needed to explain why a Calibrated Modified Robertson's Recurrence Model still results in an underestimation of minimum delay by 7%, an underestimation of minimum stops by 38%, and an underestimation of minimum performance index by 4%. Suggestions include : revisions to the form of the F

factor (making βT a real value vs an integer value), prediction downstream is based on a portion of the previous calculated interval ($i+\beta T-1$ vs $i+\beta T-2(3,4...)$).

4. Detail statistical analysis of the travel time of individual vehicles in a platoon (platoon followers) vs the travel time of platoon leaders so that a better understanding of the beta empirical parameter can be determined.

7. LIST OF REFERENCES

1. LIGHTHILL, M.J. and WHITHAM, G.B., On Kinematic Waves I, Flood Movement In Long Rivers. Proceedings of Royal Society, A 229 (1955), 281 -- 316.

2. LIGHTHILL, M.J. and WHITHAM, G.B., On Kinematic Waves II, A Theory of Traffic Flow On Long Crowded Roads. Proceedings of Royal Society, A 229 (1955), 317 -- 345.

3. PACEY, G.M., The Presence Of A Bunch Of Vehicles Released From A Traffic Signal. RRL Research Note No. RN/2665/GMP, Road Research Laboratory, January 1956 (unpublished).

4. LEWIS, B.J., Platoon Movement Of Traffic From An Isolated Signalized Intersection. Highway Research Board. Bulletin No. 1978, 1958.

5. Gerlough, D.L., Some Problems In Intersection Control. Theory Of Traffic Flow (First Symposium On The Theory Of Traffic Flow), Warren, Michigan, 1961.

6. GRAHAM, E.F., and CHENU, D.C., A Study Of Unrestricted Platoon Movement Of Traffic. Traffic Engineering, Vol. 32, No. 7, April 1962.

7. GRACE and POTTS, Diffusion Of Traffic Platoons. Proceedings Of Australian Road Reserach Board, Vol. 1, Part 1, 1962.

8. GRACE and POTTS, A Theory Of The Diffusion Of Traffic Platoons. Journal Of Operational Research Society, Vol. 12, No. 2, March 1964.

9. HERMAN and POTTS, Single Lane Traffic Theory And Experiment. Theory Of Traffic Flow, Symposium Proceedings, Elsevier, 1961.

10. HERMAN ET AL, Behaviour Ot Traffic Leaving A Signalized Intersection. Traffic Engineering And Control, Vol. 5, No. 9, January 1964.

11. HILLIER and ROTHERY, The Synchronization Of Traffic Signals For Minimum Delay. Transportation Science, Vol. 1, No. 2, May 1967.

12. ROBERTSON, D.I., Contribution To Discussion On Paper 11, Symposium On Area Control Of Road Traffic, Institution Of Civil Engineers, London, 1967, 140 -- 141.

13. ROBERTSON, D.I., TRANSYT : A Traffic Network Study Tool. Road Research Laboratory, Report No. LR 253, 1969.

14. ROBERTSON, D.I., TRANSYT Method For Area Traffic Control. Traffic Engineering And Control, Vol. 2, No. 6, October 1969, 276 - 281.

15. ROBERTSON, D.I., TRANSYT : A Traffic Network Study Tool. Fourth International Symposium On The Theory Of Traffic Flow, Karlsruhe, 1969.

16. SEDDON, P.A., The Prediction Of Platoon Dispersion In the Combinations Methods Of Linking Traffic Signals. Transportation Research, Vol. 6, 1972, pp. 125 -- 130.

17. SEDDON, P.A., Another Look At Platoon Dispersion -1 : The Kinematic Wave Theory. Traffic Engineering And Control, Vol. 13, No. 8, December 1971, 332 -- 336.

18. SEDDON, P.A., Another Look At Platoon Dispersion -2 : The Diffusion Theory. Traffic Engineering And Control, Vol. 13, No. 9, January 1972, 388 -- 390.

19. SEDDON, P.A., Another Look At Platoon Dispersion -3 : The Recurrence Relationship. Traffic Engineering And Control, Vol. 13, No. 10, February 1973, 442 -- 444.

20. HARTLEY, M.G., and POWNER, E.T., Traffic Studies At The University Of Manchester, Institute Of Science And Technology. Traffic Engineering And Control, Vol. 13, No. 6, October 1971, 256 -- 258.

21. TRACZ, M., The Prediction Of Platoon Dispersion Based On Rectangular Distribution Of Journey Time. Traffic Engineering And Control, Vol. 16, No. 11, November 1975, 490 -- 492.

22. LAM, J.K., Studies Of A Platoon Dispersion Model And Its Pratical Implication. Proceedings Of the Seventh International Symposium On Transportation And Traffic Theory, Kyoto, Japan 1977, 119 -- 144.

23. HUDDART K.W., and TURNER E.D., Traffic Signal Progression - G.L.C. Combination Method. Traffic Engineering And Control, Vol. 11, No. 7, November 1969, 320 -- 322.

24. WILLIAMS, D.A.B., Area Traffic Control In West London : Assessment Of First Experiment. Traffic Engineering And Control, Vol. 11, No. 5, July 1969, 261 -- 281.

25. HOLROYD J. and HILLIER J.A., Area Traffic Control In Glasgow : A Summary Of Results From Four Control Schemes. Traffic Engineering And Control, Vol. 11, No. 3, September 1969, 220 -- 223.

26. ROBERSON, D.I., Cyclic Flow Profiles. Traffic Engineering And Control, June 1974.

27. HAUER, E., Discussions On Platoon Dispersion Models. University of Toronto, 1973.

28. FOSTER, E.L., A Micro-computer Cyclic Flow Profile - A Research Proposal As Partial Fulfillment Of The Requirements For The Degree Of Master Of Science, University Of Alberta. March 1980.

29. RICHARAD, D., Permitted The Use Of The Skeleton Program ROBERT (ROBERTson's Platoon Dispersion Model). Metropolitan Toronto Roads And Traffic Department, March 1982.

30. GARTNER, N., Microscopic Analysis Of Traffic Flow Patterns For Minimizing Delay On Signal-Contolled Links. Highway Research Board Record 445, pp. 13 -- 15, 1973.

31. SALTER R.J., Determination Of The Effective Green Time. Chapter 35, Highway Traffic Analysis and Design, The MacMilliam Press Ltd., Great Britain, 1974.

32. TEPLY, S., Comparative Traffic Behaviour. Traffic Engineering And Control. December 1978.

33. BOLGER, D., Planning For Intersection Capacity. M.Eng. Thesis, The University Of Calgary, 1977.

34. ST. Package, Interactive Statistical Graphics Package. Computing Services, University Of Alberta, R23.0280.

35. TEPLY, S., Saturation Flow At Signalized Intersections Through A Magnifiying Glass. Eighth International Symposium On Transportation And Traffic Flow Theory, December 1980.

36. ROBERTSON, D.I., TRANSYT : A Traffic Network Study Tool. Road Research Laboratory, Report No. LR 253, 1969.

37. TRAFFIC RESEARCH CORP. SIGRID Program : Notes And User Manual. Toronto Traffic Control Centre, 1965 - 73 (unpublished).